PROCEEDINGS

THE INSTITUTION OF CIVIL ENGINEERS

PART I
MAY 1955

ORDINARY MEETING

14 December, 1954

DAVID MOWAT WATSON, B.Sc., President, in the Chair

The Council reported that they had recently transferred to the class of

Members

Andrew, Henry Stuart.
Broughton, John Henry Albert, B.Sc.(Eng.) (*London*).
Ford, Hugh, D.Sc.(Eng.) (*London*), Ph.D. (*London*).
Lamb, John Cowper, M.C.
McDougall, Archibald, B.Sc. (*Edinburgh*).
Matthews, William.
Sabin, Frank Collins, B.Sc.(Eng.) (*London*).

and had admitted as

Graduates

Ayles, James Kavanagh, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Baines, James Humphreys, B.A. (*Cantab.*), Stud.I.C.E.
Blackwell, Colin Roy, B.Sc. (*Bristol*), Stud.I.C.E.
Boon, Desmond Frederick, B.Sc. (*Durham*), Stud.I.C.E.
Bridges, James Crichton, B.Sc. (*Glasgow*), Stud.I.C.E.
Callaghan, George Denis, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Cannon, John Charles, B.A. (*Cantab.*).
Christie, Ian McCallum, B.Sc. (*Glasgow*).
Clifford, John Edward, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Colloff, Geoffrey, B.Sc. (*Birmingham*), Stud.I.C.E.
Corney, Gerald Walter, B.Sc. (*Wales*), Stud.I.C.E.
Cowley, Ralph Alan, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Crow, Geoffrey Philip, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Cumming, Stanley, B.Sc. (*Belfast*), Stud.I.C.E.
Darch, John, Stud.I.C.E.
D'Lord, Joseph Peter, Stud.I.C.E.
Donkin, Alfred Walter, B.Sc. (*Durham*), Stud.I.C.E.
Edwards, Graham Baldwin, B.Sc. (*Wales*).
Edwards, Paul Harper Dickinson, Stud.I.C.E.
Elliott, Christopher Dennis, B.Sc. (Eng.) (*London*), Stud.I.C.E.

English, Charles John, B.Sc. (*Cape Town*), Stud.I.C.E.
Fardon, Reginald Stuart, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Fisher, Alan Geoffry, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Fordham, Francis Myrddin, B.Sc. (*Wales*), Stud.I.C.E.
Fraser, Angus Stuart, B.Sc.(Eng.) (*Natal*).
Giddings, Kenneth, B.Sc. (*Manchester*), Stud.I.C.E.
Glen, Charles, Stud.I.C.E.
Grieve, John, Stud.I.C.E.
Gulliver, James Gerald, B.Sc. (*Glasgow*).
Hadjigeorgiou, John, B.Sc. (*Birmingham*), Stud.I.C.E.
Han, Eu Sin, B.Sc. (*Rangoon*).
Hansed, Jack, B.Sc. (*Birmingham*).
Harley, Bernard Victor, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Hibbert, William Arthur, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Ho Thian Hock, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Holden, Frank, B.Sc. (*Manchester*).
Hughes, Joseph Brian, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Hunt, Herbert Arthur, B.Sc.Tech. (*Manchester*).
Jacob, Peter Hawkins, B.A. (*Cantab.*).
Jones, Charles William, B.Sc.Tech. (*Manchester*), Stud.I.C.E.
Kennedy, Quintin Byron, B.Sc. (*Durham*).
Knight, Philip Gordon, B.Sc. (*Manchester*).
Kolek, Jan, B.Sc.(Eng.) (*London*).
Lewis, Charles David, B.Sc. (*Wales*).
Macfarlane, Alexander, Stud.I.C.E.
McKenzie, John Cormack, M.Sc. (*Queens*), M.A., M.A.I. (*Dublin*).
MacLeod, Malcolm Donald, B.Sc. (*Aberdeen*), Stud.I.C.E.
Mandy, Gordon William, Stud.I.C.E.
Marten, Graham Charles Woodcock, B.Sc.Tech. (*Manchester*).
Moini, Syed Qamruddin, B.Sc. (*Aligarh*).
Muller, Michael Hugh Sigvald, B.A. (*Cantab.*).
Newsome, David Harry, B.Eng. (*Sheffield*), Stud.I.C.E.
Nutt, Geoffrey William, Stud.I.C.E.
Panesar, Ripdaman Singh, B.Sc. (*Wales*).
Parsons, Geoffrey Frank, Stud.I.C.E.
Phillips, William, B.Sc. (*Durham*).
Phillips, William Freer Coles, B.A. (*Cantab.*), Stud.I.C.E.
Pigott, Pierce Thomas, B.E. (*National*).
Poole, David Allen, Stud.I.C.E.
Quenet, François John, Stud.I.C.E.
Rendall, Gordon Campbell, B.Sc. (*Durham*), Stud.I.C.E.
Rollason, Peter James, B.Sc. (*Birmingham*), Stud.I.C.E.
Rutherford, Clive Cowan, B.Sc. (*Durham*), Stud.I.C.E.
Ryder, Alan Cunningham, Stud.I.C.E.
Saif, Qadir Jabbar, B.Sc. (*California*).
Schofield, Andrew Noel, B.A. (*Cantab.*), Stud.I.C.E.
Shasha, Fouad Abdulla, Stud.I.C.E.
Sherwood, Edwin, B.Sc. (*Durham*).
Singh, Kanwar Yaduveer, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Smith, George Bodie Thomson, Stud.I.C.E.
Sprott, William Christopher, B.Sc. (*Edinburgh*), Stud.I.C.E.
Steele, John James, Stud.I.C.E.
Sturrock, Kenneth Robertson, B.Sc. (*St. Andrews*).
Walker, Leslie James, B.Sc. (*Edinburgh*), Stud.I.C.E.
Wallace-Jones, George Maurice, B.A. (*Cantab.*).
Ward, James, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Watkins, Albert Colin, B.Sc. (*Wales*), Stud.I.C.E.
West, Frank Ernest, Stud.I.C.E.
Wilson, Ian William, B.Sc.(Eng.) (*London*).
Woodlock, Victor John, B.Sc.(Eng.) (*London*).
Young, Ian Victor, Stud.I.C.E.

and had admitted as

Students

Akuruka, Godwin Akujobi.
Ardley, Alan Reginald.
Ashley, Neil.
Balfour, Hugh Crawford.
Banerjee, Dilip Kumar.
Bayley, Richard Colin.
Bigby, Peter Graham Murray.
Blackwood, Derek Smith.
Bruce, William Allenby.
Campbell, James Maurice.
Castle, Norman Frankland.
Cathcart, David Strain.
Chester, John Richard.
Chua Thye Guan.

Clark, Jackson Abadee.
Clemow, Vaughan Spencer.
Constance, Graham Michael.
Cornwell, Kenneth.
Craig, Kenneth David.
Curry, David George.
Curtis, Keith Anthony Hadden.
Daniel, Michael Olorunnaiye.
Dassenaike, Earle Brian.
Davis, David Michael.
Dembrey, Roger Martin.
Douglas, Murray King.
El-Miqdadi, Usama Darwish.
Endean, Peter Walwin.
Foy, Thomas Alexander.
Fulton, Hugh Moore.
Gaynor, David Donald Emerson.
Gorwala, Feroze Dadabhoy.
Gunawardana, Olsen Amarasiri.
Haywood, George Michael.
Hemmin, Ian Walter.
Henry, Keith Iain Malcolm.
Hewett, Bruce Ogston.
Home, Kenneth.
House, Trevor Moore.
Husband, Anthony.
Jackson, John Francis.
Jarvis, Ian William Hunter.
Jarvis, William Hardie.
Jelly, Fullerton McWilliam.
Jones, Arfon Harry.
Jose, Thomas, David.
Kelly, Jack Wesley.
Langridge, William Dennis.
Lawson, Walter Ian.
Lees, John Rutherford.
Lees, William Lowry, (*Jun.*).
Lillico, David Baxter.
Lim Wee Kiat.
Loukakis, Loukas.
Lyttle, Ronald Patrick Trevor.
McAlpine, William Irvine.
McCammon, Norman Robert.
McCann, Gerald Ferrie.
McCann, Owen.
Maclean, Robin Murdon.
Madden, Gerald George.
Martin, Walter George Gardner.
Martyn, Nolasco Evaristus Ransford.
Mellor, Malcolm.
Mian, Muhammad Hassan.
Mildenhall, Henry Seymour.
Miller, James.
Mistry, Jamshed Burjorji.
Monk, Reginald George.
More, Andrew.
Morris, Colin Ronald.
Munro, William Donald.
Myint, Hla.
Neden, Andrew.
Norrish, John Michael.
Odedairo, Ebenezer Olufunso.
O'Neill, John Gerard.
Osborne, Ivan William.
Paosila, Kittinand.
Pedersen, John Dam.
Pelton, Foster.
Porteous, David Stewart.
Rendall, Dennis Edgar.
Rifkind, Cecil Mosely.
Robinson, Dennis Malcolm.
Robinson, John Anthony.
Rose, Peter John.
Ross, John Macleod.
Senyard, Michael Howard.
Shah, Jitendra Becharlal.
Shuttler, Richard Montrose.
Simmonds, Brian Osborne.
Sims, Peter Gardner.
Slater, John Theodore Alan.
Smith, Neville John Christopher.
Steele, Peter Robertson.
Stephenson, Frederick William.
Stone, Peter John.
Swanson, Norman John.
Tan Beng Lew.
Tang, Basil Clinton.
Thackray, John Edwin.
Thexton, Alec Frederick.
Thomson, Peter.
Toufaha, Sami Ahmad Dawoud.
Trapp, Michael Royston.
Tricker, John.
Turner, Malcolm Stewart.
Usman, Mohamed Tukur.
Wardle, David Gordon.
Warren, Roy Benjamin.
Watson, Roy.
Watson, William Scott.
Weeden, Michael Henry Dennis.
Westley, John William.
Weston, Brian Sydney.
Whitcombe, Allen.
White, Peter James.
Whitehouse, John Patrick.
Willett, David Charles.
Willment, Roger Edward.
Wiseman, Martin Hubert.
York, David.

THE UNWIN MEMORIAL LECTURE, 1954

The President reminded members that in 1947 Miss E. T. Unwin, a niece of the late Dr W. C. Unwin, Past-President of the Institution bequeathed a sum of £2,500 to the Institution. The conditions attachin to this legacy were that the sum of £1,500 was to be invested in trust fo the foundation of a " William Cawthorne Unwin Lectureship " consistin of an Unwin Memorial Lecture on Engineering Research to be delivere to the members of the Institution annually, or at such other period c recurrence as the Council might determine. The remainder of the legac was for the endowment of a section of the Institution Library.

The Council had this year requested Dr Lea to deliver the fifth Unwi Memorial Lecture. The President said Dr Lea had been Director of th Building Research Station since 1946 and was well known for his work o cement, as well as for other work.

Dr Lea then delivered his Lecture.

EXPERIMENTAL SCIENCE AND CIVIL ENGINEERING RESEARCH

by

Frederick Measham Lea, O.B.E., D.Sc., F.R.I.C., Hon. A.R.I.B.A

When the Council of the Institution did me the honour of asking me t give this Unwin Memorial Lecture, I felt in some doubt as to my qualifica tions. Those who have delivered the four previous lectures have bee distinguished engineers and of the first two, Dr Oscar Faber could claim personal link with Unwin, whilst Professor A. J. S. Pippard is a success to his Chair. As one trained in the pure sciences I cannot speak to yc as an engineer, but both from heredity and subsequent experience it ha been my fortune to have had many contacts with civil engineerin Though I strayed from a family tradition of engineering when I first turne to pure science, I soon found myself in contact with engineering when joined the staff of the Building Research Station and this has remaine and developed for nearly 30 years. I might perhaps claim some lin even though a very tenuous one, with Unwin in that my uncle, the la Professor F. C. Lea, received his first university appointment in 1900 chief assistant to Professor Unwin at the City and Guilds Institute.

I was stimulated when I read E. G. Walker's biography of Unwin think that, if I could not speak as an engineer, I might perhaps speak a subject—the influence of Scientific Developments on Civil Engineerin Research—which would have come close to his heart and which, if it d

not hit the bull's-eye, might at least not miss the target, in terms of the subject laid down in the bequest establishing this lectureship.

If I read the life of Unwin correctly the great influence he exerted on the progress of engineering came from his application of scientific principles to engineering design. Unwin's aim was the practical application of the results of scientific study and the understanding of "rules-of-thumb" through the reasoning that underlies them.

In my title I have used the term "Civil Engineering Research," but I shall confine myself to the more limited field of structural engineering. I do this because it is here that my contacts with civil engineering have lain, but I am conscious that some other lecturer, with a knowledge wider than mine, could draw many examples from other fields.

Whatever definition we may adopt on the scope of research on structures it should include the study of the properties of engineering materials and of the behaviour of full-scale structures or parts of them, and the application of the results to the theory of design, to practice, and to regulations.

Research in the pure sciences on the properties of materials is usually directed to substances that are pure, or of known and controlled composition. This simplification is not possible for the engineer who has to deal with the inevitable variations in manufactured materials or the wide variability of natural ones. The theoretical basis of any engineering design usually requires some idealization of the properties of materials, but the nearer the assumed characteristics represent the real properties the more applicable is the theory. Much civil engineering research in universities and in the Department of Scientific and Industrial Research during the past 30 years has been devoted to a closer assessment of the properties of common engineering materials such as concrete, steel, timber, ınd brickwork. Attention in particular has been directed to the more obscure factors, as for example, creep and shrinkage of concrete, and the effects of impact and fatigue.

A very noteworthy example is to be found in soil mechanics where the mprovement in knowledge of the mechanical and physical properties of oils has formed the basis of the major advances made during the past 30 ears. None of the theories of the behaviour of soils advanced before the nd of the nineteenth century took any account of the mechanical interction between the solid soil particles and the water contained in the voids. 'hese older theories assumed properties so remote from reality that their ıfluence on engineering practice was negligible. It was only when the ıeories of soil behaviour were modified to conform with the real properties f soils that theoretical methods of design became of value to the practising ıgineer.

We find a parallelism in the field of structural design. Theoreticalesign procedures involve not only an idealization of the properties of the aterial but also a simplification of the mechanism of the behaviour of the semblages of materials which make up an engineering structure. It was

this that led Unwin to comment in his 1911 Presidential Address that "the full developments by the 19th-century mathematicians of the theory of elasticity, though involving the most strenuous efforts of the highest mathematical genius, and valuable as is all definite knowledge, were for the most part not of practical importance to the engineer. Practical insight into what was important, and what was negligible, was an essential element in an engineering problem where knowledge of properties could only be partial and incomplete." If today we find that more complex mathematical treatments are of increasing value to the engineer, it is because of the great increase in knowledge of the real properties of materials and of the behaviour of structures. It reflects the information that has come, not only from laboratory investigations, but from observations on large-scale models and full-scale structures, and which enables theoretical forecasting and actual behaviour to be compared.

My purpose this evening is to consider a few of the developments in experimental science that have contributed to modern advances in civil engineering research and to illustrate some of their applications.

In research on structures the measurements that are made relate mainly to deformation or strength. Whilst strength tests may be made in the laboratory on structural elements even up to the full-scale, it is only rarely that tests to destruction can be made on full-scale structures. Deformation measurements, on the other hand, are ubiquitous and can be made equally on full-scale structures in the field or on laboratory specimens.

It is a commonplace to say that developments in any branch of pure or applied science undergo a spurt when new techniques of measurement become available, or new theoretical concepts are introduced. Civil engineering has benefited from this in many ways unforeseen at the time of the original discovery. J. J. Thomson's discovery of the electron in 1897 could not then have seemed likely to find application in civil engineering research, but the chain of events that this set in motion has led directly to some of the modern methods of measurement. If the chain has many links it is nevertheless a continuous one leading from the primitive thermionic valve of Fleming and de Forest to the most modern developments in the science of electronics. The cathode-ray tube, for example, is one of the most versatile tools at the disposal of the engineer for measurement of transient or steady strains.

The application of X-rays to the detection of flaws in metals is too familiar to need more than passing mention, but we should underestimate its importance to engineering progress if we ignored the influence of X-ray crystal analysis on the development of materials and particularly of metal alloys. The knowledge it has yielded of the crystal structure of alloys has been both a pointer and a guide to the development of new materials. We shall have cause later in this lecture to look for a moment at the application of X-ray investigations to cement and concrete.

Radioactivity when first discovered was of intense interest to the

chemist and the physicist, but its practical use in civil engineering research was remote. Today we find the use of gamma rays and neutrons developing as tools of engineering measurement.

Equally important as the influence of new scientific developments has been a converse action. The need for new or improved methods of measurement has stimulated the application of well-known principles of classical physics in the form of reliable instruments suitable for engineering measurements in the laboratory and the field. Both new principles and applications of old ones have now rendered possible quantitative measurements of phenomena that were previously inaccessible or at best only crudely measurable.

The direct measurement of displacement by gauges of one sort or another, and by extensometers in which the movement is multiplied by optical and mechanical levers, is too familiar to need any comment. I propose, therefore, to mention only the application of some other old physical principles and the utilization of some newer ones.

In principle, any physical property of a material that is changed by deformation can form the basis of measurement of strain. The changes in electric resistance, or in the natural frequency of vibration of a wire, with changes in tension, have long been known, but their development into useful and reliable methods of measurement has come only relatively recently. The need for them was the spur that led to their development. Electrical capacitance and magnetic induction are other well-known physical phenomena that have been similarly utilized and, also, to some degree, magnetostriction, the change in volume of a magnetic material when it is placed in a magnetic field. The piezo-electric effect by which certain substances, when subjected to pressure, develop an electric potential between different crystal faces, forms the basis of an invaluable method for measuring high-frequency vibrations. An early example of its application in civil engineering research may be found in the measurements made by Glanville and Grime of the passage of pressure waves in a concrete pile during driving. The reverse effect we see in the electro-mechanical transducer in which the change of dimensions of crystals, such as those of Rochelle salt and barium titanate, on application of an electric field, is used to generate pressure waves in a solid. The development of electronics has now provided means of producing high-frequency voltage pulses of adequate power and methods of measuring them. This combination of modern electronics and the classical physical principle of piezo-electricity has thus given the ultrasonic method of testing in which the velocity of propagation of a pulse of high frequency waves through a solid is measured. As applied to concrete it can be used on a laboratory specimen, or on in-situ mass concrete up to 40 ft in thickness. Though much remains to be done, we have here a weapon that enables the engineer to probe into the interior of a concrete mass.

It will not have escaped your notice that with all the means now at our

disposal we are still limited in general to the measurement of strain and only indirectly of stress. Though the engineer would much like to be able to measure stress directly, this is rarely possible in solids and even the so-called stress gauges depend on a deformation to bring their mechanism into play, resulting almost inevitably in some inequality in stress between the measuring instrument and the solid on which the measurements are made. The major exceptions are fluid gauges in which the pressures inside and outside the gauge can be balanced. Such gauges can be used very successfully, as we shall see later, for the measurement of the pore-water pressure in soils, but they are ill-adapted for use within solid bodies. If future developments should give us a direct and reliable method of measuring stress in solids, yet another very important weapon will have been added to the engineer's armoury.

The importance of new methods of measurement is well illustrated by present trends in civil engineering research. The engineer needs first to know the properties of materials. The study of these, and of the individual components of structures, has long been the concern of research and testing laboratories. These give the essential data that the engineer applies in the design of structures, checking by conventional assumptions that the stresses or deflexions in each member do not exceed certain permissible values for the assumed working loads. There is now developing, as you know well, a new approach to design based on the properties of a structure as a whole and not merely on the analysis of its individual parts. Its future development requires much more information, not only on the properties of structures as a whole, but also of the more subtle properties of materials. Whilst the newer methods of measurement have been very valuable in laboratory investigations, they have often been essential to studies of actual structures. The ability to measure transient effects produced by impact or dynamic loads and the relative ease with which appropriate gauges, recording at a distance, can be fixed to structures to give permanent records of changes during long periods of time, now permit of observations of a type that previously were often impracticable. The importance of the study of actual structures and of the relation between design calculations and actual stress distribution is illustrated by the action of the Institution in arranging for a conference to be held next autumn on "The Correlation between Calculated and Observed Stresses and Displacements in Actual Structures."

I have so far passed rapidly over some of the general ways in which physical principles—old and new—have been adapted to provide reliable measuring instruments. In the remainder of this Lecture I should like to give some illustrations of their use in engineering research. In so doing I propose to draw first on some of the work carried out by my colleagues in the Engineering and Soil Mechanics Divisions of the Building Research Station. For some examples I must go farther afield, in Britain and

abroad, for it is not given to any single organization to cover more than a fraction of these interesting applications.

The structural research at the Building Research Station has mainly been concentrated in recent years on a number of well-defined lines. Naturally the techniques of measurement used have varied with the nature of the problems and the kind of information sought, but on the purely instrumental side perhaps the Station's major contribution has been the development of the vibrating-wire gauge from the earlier forms in which it originated in France and Germany.

It will, perhaps, assist in giving a background to the illustrations I have selected if I first outline very briefly the scope of the structural research work of the Building Research Station. This I can conveniently do by grouping it under a few major heads.

The strength of structures as a whole

It has long been realized that a framework is much stiffened by floors and walls, and by concrete encasement of steel beams and stanchions, but apart from certain allowances for the latter, no quantitative data have been available which would permit of advantage being taken of this composite action in design. We have, therefore, been much concerned with the structural interaction between the various elements of a structure including those normally regarded as infilling or claddings. The investigations include both laboratory studies and tests on full-scale structures. Examples of the former are the composite action between brick walls and their supporting reinforced concrete beams, between steel frames and various brick and block infillings and sheet wall claddings, and the effect of concrete encasement on steel joists. The studies on whole structures have included the detailed observations of the stresses in the steel beams of a large building, starting with the bare beams before erection and following through all stages of construction up to final overload tests on the floors. As a corollary to this work, attention has also been given to the design of steel frames.

Investigations on bridges

The studies on bridges fall into two parts. The first has been the assessment of the load-carrying capacity of existing cast-iron girder and masonry-arch road bridges and of the dynamic stresses produced by impact. The second has been concerned with the design of reinforced concrete bridge deck slabs for slab and girder systems and has been extended to include jack-arch and filler-joist systems. This second part of the investigations is closely related to the studies of composite action that I have already mentioned. It has involved laboratory studies on $\frac{1}{3}$-scale models and measurements on some recently constructed bridges.

Prestressed concrete

Investigations on prestressed concrete have been concerned both with details of the manufacturing process and the properties of prestressed members. In the latter particular attention has been given to the loss of prestress caused by creep of the steel and shrinkage of the concrete and to the impact strength and fire resistance. The loss of prestress in particular has been followed in field studies.

Plain and reinforced concrete

The major objects of attention in plain and reinforced concrete work in recent years have been load-bearing concrete walls, shell-roof construction, and a revived interest in certain problems of mass-concrete construction. Tests on concrete walls were initiated to provide basic information for the guidance of designers of multi-storey buildings and were made on full-size walls and model walls to determine the effects on wall strength, deformation under load, and mode of failure, of varying the wall dimensions and reinforcement in cases of axial and eccentric compression, and of introducing door and window openings. The mathematical analysis of the stresses in shell roofs has received considerable attention in recent years, but few experimental data are available on the real behaviour of such structures. The analysis is particularly likely to be inaccurate after cracking of the concrete has occurred and an investigation is being made of the behaviour of shell roofs at all loads up to failure. The work on mass concrete has been mainly concerned with deformations arising from creep, shrinkage, and thermal effects ; in particular, a study has been made of the likelihood of cracking of certain types of concrete dams.

Soil-mechanics problems

The soil-mechanics work of the Building Research Station falls into the three main branches :—foundations, earthworks, and earth pressures. In all branches of this work field studies, associated with experimental laboratory investigations, play a large part.

As examples of the studies on foundations, I may mention :—the work on house foundations on shrinkable clays to design against the effects of seasonal shrinkage arising from the depletion of the soil water by drying and by growing vegetation and trees ; observational work on the settlement of large structures and the bearing capacity of piles ; and the special problem of founding structures on deep beds of soft compressible soils.

The problems of the stability of earthworks have included those arising in sea-defence banks, where, for example, particular attention has been given to the modes of failure of the banks in the 1953 East Coast floods and the lessons to be drawn from them. Studies on pore-water pressures and seepage have been important items in this work and also in connexion with the construction of earth dams.

An interesting development in earth pressures has been the investiga-

tion of the pressure on tunnels for which design still rests largely on traditional procedures and experience. Pressures on sheeted excavations is another subject worthy of mention.

To make this summary less incomplete may I add that work is also proceeding on subjects such as vibrations in buildings, concrete pipelines, expansion joints in buildings, and on the general engineering properties of structural materials.

From this variety of subject matter I have selected a few illustrations to show how the development of some of the experimental techniques I have mentioned has facilitated research.

In field tests on bridges the vibrating-wire strain gauge has been much used where remote measurements were desired during a long period of time, for this gauge is very stable and is free from shifts in zero reading in contrast to the resistance gauge which is not stable during long periods. The latter has, however, an advantage for dynamic tests for the analysis of the results is much more simple.

Both types of instrument were used in an investigation of the dynamic stresses in cast-iron girder bridges when subject to the load of a moving vehicle. The main object was to determine the peak value of the strain produced at mid-span of a girder as a vehicle crossed the bridge and to compare this with the strain under static loading; from such comparisons it was hoped to obtain a sound basis for recommendations as to the "impact allowance" to be made in the design of girder bridges generally.

Two series of tests were made. In the first, the strain gauges were of the vibrating-wire type shown in Fig. 1. The gauge consists of a thin steel wire stretched between the knife edges, one of which is free to move longitudinally. The wire is maintained vibrating at its natural frequency by an electrical method. The knife edges are held firmly against the girder under test, and a change of strain in the girder alters the tension in the wire and hence its natural frequency. The gauge is used in combination with a reference instrument of fixed frequency; electrical impulses from both instruments are superimposed to produce beats having a frequency equal to the difference between the frequencies of the two instruments. Changes in the frequency of the test gauge caused by variations in strain result, therefore, in identical changes in the beat frequency.

The joint output from the two instruments was applied to the plates of a cathode-ray oscilloscope, leading to an oscillation of the electron beam with a frequency equal to that of the higher of the two applied frequencies and with an amplitude which increased and decreased with the same frequency as that of the beats. A photographic record was made, as in Fig. 2 (facing p. 256), by photographing the deflexion of the spot at the end of the cathode-ray tube on a moving film. The trace was in the form of a series of waves, each wave being the envelope of the traces of the high-frequency oscillations, the distance between successive waves being inversely proportional to the beat frequency.

The cathode-ray tube used was of the double-beam type, the second beam being used to record a continuous wave of frequency 50 cycles per second, produced by a battery-driven alternator. This wave was used as a time base.

In order to locate the position of the vehicle at any time, and to provide a method of determining its speed, photoelectric cells and relays were set up at various points on the bridge; these were operated by the action of boards, projecting from the vehicle, which obscured the daylight falling

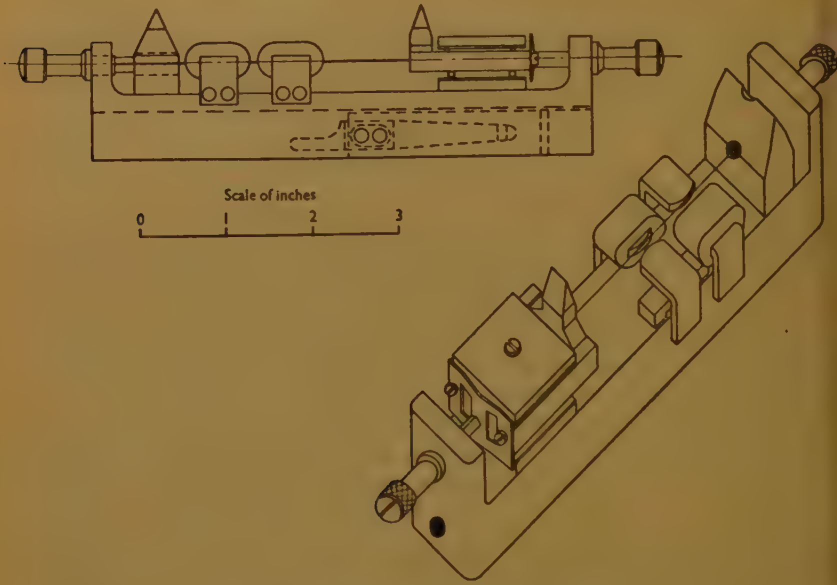

FIG. 1.—VIBRATING-WIRE STRAIN GAUGE

on the cell as the vehicle passed the selected point. When each cell relay came into operation, it caused the 50-cycle trace on the oscilloscope screen to be interrupted; one such interruption can be seen in Fig. 2 (facing p. 256).

Analysis of the results of such tests involves the measurement of large numbers of traces in order to determine the beat frequencies and, hence, the strains in the girder. In a second series of tests this considerable task was eliminated by using electrical-resistance strain gauges fixed to the girders. Such gauges are particularly suitable for dynamic measurements because of their lack of inertia and because they enable a continuous direct recording of strain to be made. The gauges were connected to an amplifier and the resulting changes in potential, owing to variations in changes of strain in the girder, were applied to the plates of the oscilloscope. The

movements of the electron beam were recorded as in the first series of tests.

A second oscilloscope and camera were used for recording the trace of a sine wave of known frequency, and this trace was broken momentarily when photoelectric cells were operated as the vehicle passed selected points on the bridge. A typical strain trace and its associated time trace is shown in Fig. 3. Movement of the spot in each trace is from left to

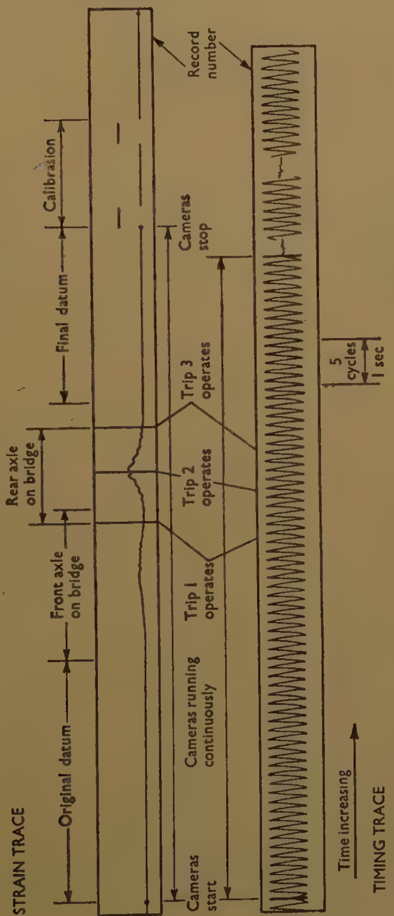

FIG. 3.—WIRE-RESISTANCE GAUGE: STRAIN AND TIMING TRACES FOR A VEHICLE PASSING OVER A BRIDGE

right. Both cameras start together and the left-hand spot on the strain trace and the vertical line on the timing trace correspond with this moment. The cameras then run continuously until the beginning of the first calibration mark, though the records are not the same length because the cameras run at slightly different speeds.

Such tests have shown that the "impact factor" commonly used in bridge design is often unnecessarily high. The actual ratio of dynamic stress to static stress for a particular vehicle was found to range between about 0·8 and 1·3 and was usually not far removed from unity. It appeared that bounce of the vehicle was an important cause of increased stress under dynamic loading, and measurements were, therefore, included of the natural frequency of the vehicle. Such measurements were made with a Rochelle salt crystal pick-up and oscillograph and camera. This pick-up was of the inertia type, consisting of a crystal of Rochelle salt supported at three corners, the fourth being free. When the pick-up is fixed to a vibrating body, the crystal bends and this bending produces a voltage difference between top and bottom surfaces of the crystal which is proportional to acceleration. The resulting trace on the moving film in the camera can be used for the determination of frequency.

Another investigation, that on the stiffening of frames by floors and walls, affords an example where modified forms of measuring instrument needed to be developed. In order to obtain data on the structural interaction between the various parts of a building, measurements were made of the stresses in some of the beams and stanchions in the steel frame of a large office building in London. It was thought desirable to determine the complete stress history for these members, during construction, during special loading tests, and during a period of years for normal conditions of use. The strain gauges to be used had to be stable for long periods, resistant to the impact and vibration associated with construction of the building, and suitable for burying within the concrete encasement of the steelwork. Since large numbers of gauges were to be used and it would be impracticable to recover most of them, the design had to be simple and cheap.

None of the gauges previously used immediately satisfied these requirements. From the point of view of stability and reliability, only the vibrating-wire gauge appeared to be suitable; but the form in which it had been developed for bridge tests was too expensive and not sufficiently robust for a long-term investigation on stresses in buildings. Accordingly, a new form of vibrating-wire gauge was specially designed. In order to avoid the risk of slipping of the knife-edge mounting, the wire was stretched between two steel posts screwed into prepared holes in the structural member. With this type of fixing, the gauge length is not so precisely defined as is possible with knife edges, but the error in strain estimation is of negligible importance. The modified strain gauge is shown in Fig. 4. The posts are fixed at 6-in. centres, and are firmly held in position by lock-nuts. The wire, of 0·018-in. dia. passes through a hole in each post and is fixed by means of a tapered pin driven into a second hole at right angles to that for the wire. The arrangement of holes, wire, and pin is such that the driving home of the pin firmly grips the wire, squeezing it by about one-third of its diameter. In mounting the gauge one end of the wire is

first fixed; the wire is then stretched by a calibrated spring to a predetermined stress before the second end is fixed. The free vibrating length of wire is accurately measured for each gauge.

All the components were made from a special stainless steel. The wire had to be magnetic, rustless, and free from creep at its working tension. In operation the wire is plucked by discharging a condenser through a

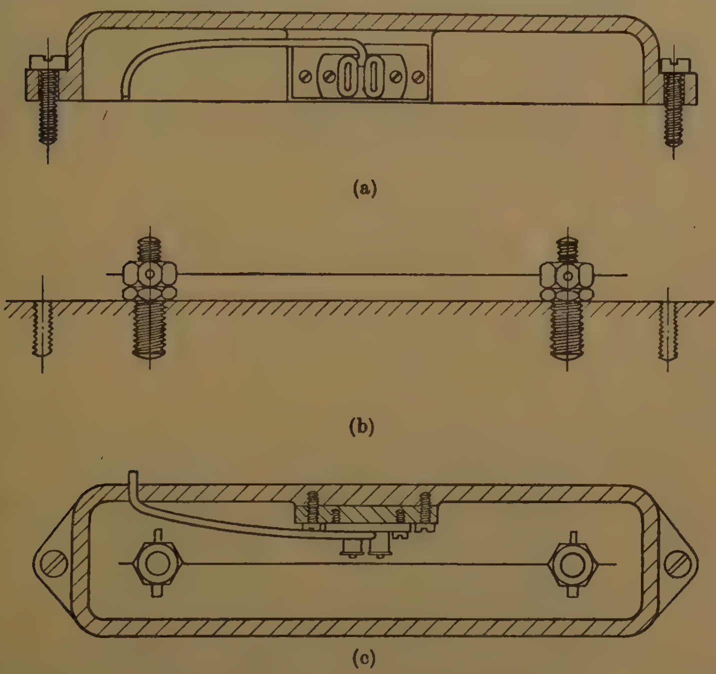

FIG. 4.—MODIFIED VIBRATING-WIRE GAUGE WITH CAST-IRON COVER FOR USE ON STEEL BEAMS TO BE ENCASED WITH CONCRETE

small electromagnet held close to it; the same electromagnet picks up the vibration and the oscillating electric potential is amplified and fed to earphones or a cathode-ray tube. Comparison of the vibrations with those of a reference gauge is made aurally by the elimination of beats, or visually by a study of the Lissajous figures produced by the test gauge and the reference gauge on the screen of the cathode-ray tube.

For gauges buried in concrete, the electromagnet is mounted in a heavy cast-iron cover, which protects the gauge during construction of the building and keeps out dirt and damp. The cover is made watertight by sealing the base with a bituminous compound. The leads from the electromagnet are taken to a convenient measuring station. Since the measurements are concerned solely with frequencies of oscillations there is no need to take special precautions with electrical connexions and the switchgear

used for a large group of gauges can be very simple. In this respect, the vibrating-wire gauge has an important advantage over the electrical resistance gauge.

Gauges were fixed on steel beams during erection of the building. Initially sixty gauges were fitted to beams and stanchions in the basement to study the effect of settlement of the building. Later a further ninety-two gauges were fitted to beams at third-floor level to study the composite action between the beams and their encasement and the floor slabs. The results will be described fully elsewhere, and it suffices to mention here that the tests have shown the great importance of composite action and the need for further work, both in the laboratory and on actual structures, to obtain data so that proper allowance can be made for these effects in design.

A similar technique for measuring strains was used in an investigation of the behaviour of the prestressed concrete main beams of a London building. The principal object was to determine the loss of prestress that occurred during a period of time owing to shrinkage and creep of the concrete. The deformations were, however, considerably greater than those measured in the steel framework of the previous example and hence a higher working tension was used for the vibrating wire. Because of this, a silver-plated piano wire was used after tests had shown that it did not creep under even higher loads than those to which the gauge wires would be subjected.

Another development in the use of the vibrating-wire gauge is afforded by the measurements made of the loads and of the changes in diameters in tunnel linings. Here a gauge was required which would function satisfactorily for long periods and under water.

In a new type of lining for a water-pressure tunnel being developed by the Metropolitan Water Board, the lining is not watertight and the water is retained by the surrounding clay. The problem was to measure, on an experimental length of tunnel, the degree to which the external load was removed from the lining as the head of water was increased and in the limit to estimate the safe maximum head without risk of collapse of the lining. Here gauges were required which would function satisfactorily for several years, under water, and could be read remotely. One type of gauge had to measure the circumferential load in the lining and another had to measure the change in tunnel diameter under a water-head of 140 ft.

The load gauge (Fig. 5) consists, very simply, of a cylindrical steel tube which is compressed axially by the load and changes the frequency of a vibrating wire held along the tube axis. The whole of the load in one tunnel ring is carried by the two load gauges, as shown in Fig. 6. The effect of external water pressure and of non-axial loads on these gauges is small and they have proved quite stable during a period of 3 years.

The diameter gauge also used a vibrating wire and is illustrated in Fig. 7. The gauge, in a cylindrical watertight case, is clamped to one wall

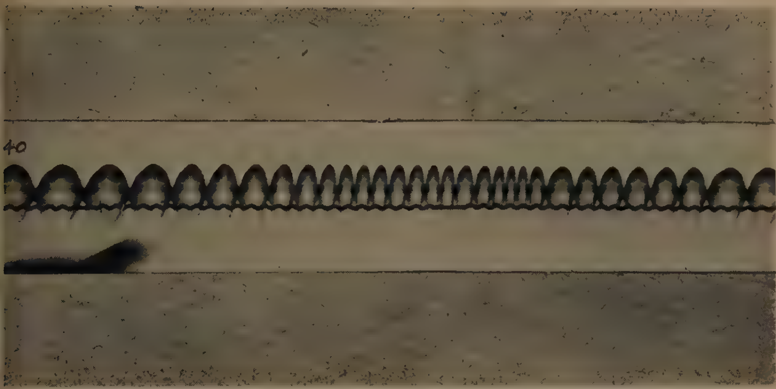

Fig. 2.—Vibrating-wire strain gauge. Record of beat frequency

Fig. 6.—Cylindrical vibrating-wire load gauges measuring circumferential thrust in tunnel lining

Fig. 8.—Equipment for measurement of pore-water pressure

Fig. 10.—Electron microscope photograph of set cement
($\times$ 20,000)
(Photograph by Mme A. Oberlin, Laboratoire de Minéralogie, Paris)

of the tunnel and a wire W extends to the opposite wall. This wire is kept in almost constant tension by a long highly compressed spring C. Axial movement of the wire W relative to the gauge body is transferred through a watertight metal bellows to a tension spring T and to the vibrating wire V inside the gauge. The sensitivity of the gauge is reduced and the range of movement increased by reducing the stiffness of spring T. Measurement to an accuracy of 0·001 in. was found possible in the tunnel. The large effect of external water pressure on this gauge is eliminated by connecting it to an air bell anchored to the tunnel invert.

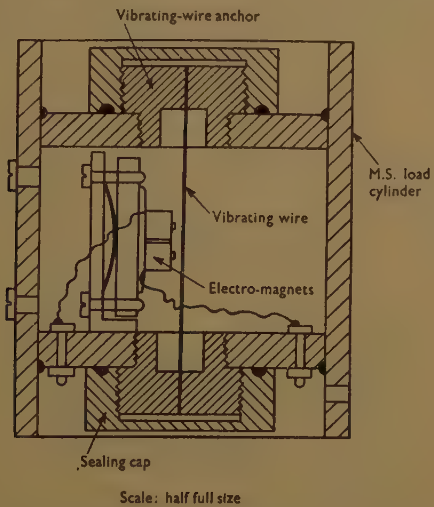

FIG. 5.—SUBMERSIBLE VIBRATING-WIRE LOAD GAUGE

The laboratory study of soils during the past two decades has led to the development of various experimental procedures, now well known, for the determination of the physical properties of soils both in the laboratory and in situ. A particularly valuable and relatively recent development is the technique for measuring pore-water pressures, for it gives an insight into what is happening in the middle of a soil deposit that cannot be obtained by any other means. It will be well known to you that when a load is applied to a saturated compressible soil the additional stress is first carried by an increased pressure of the water in the pores. As water drains away the pore-water pressure dissipates and the additional stress is transferred to the soil grains resulting in an increase in the shear strength of the soil. Thus, by following the change in pore-water pressure the increase in strength of the soil can be assessed. This is important in stability problems such as those that arise in the construction of earth dams, particularly if the available material is fine-grained and rather impermeable.

A satisfactory field equipment for the measurement of pore-water pressures has been developed and its main features can be seen in Fig. 8.

The piezometer point consists essentially of a porous grindstone disk (2 in. dia., ⅜ in. thick) set in a moulded polythene case. Two polythene tubes (3 mm internal dia.) run from the point to a Bourdon gauge. The whole system is filled completely with water and the pressure in the water is given by the Bourdon reading. In practice the piezometer point is laid face downwards at the desired position in the body of the dam and the polythene tubes are laid in a trench dug across the surface of the fill, which is then carefully backfilled with selected material. The two tubes go into a gauge house, sited at a convenient position outside the dam, where the

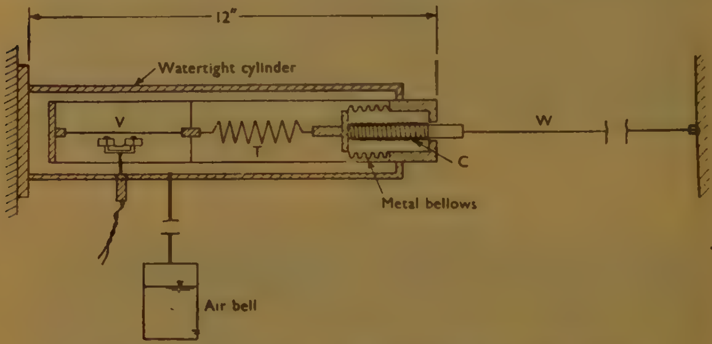

FIG. 7.—SUBMERSIBLE VIBRATING-WIRE EXTENSOMETER

instrument board has one Bourdon gauge for each point. Special provision is made for de-airing and filling the system with water.

Fig. 9 shows some typical results obtained in an investigation on an earth dam. The lower figure shows the position of three piezometer points in relation to the dam section; the upper figure shows results obtained with one of the piezometer points. In this figure the lower curve shows the level of the fill over the point and its progress with time of construction; the curve above it shows the water pressure resulting from the weight of the placed fill and how it varies with time. At the top of the figure a curve shows the pore-water pressure expressed as a percentage of the fill pressure and shows how it decreases with time. In this case the value was 100% in the early stages which led to a modification being made in the dam design. The pore-water pressure readings were then used as a control on the rate of fill placement for the remainder of the construction period. A similar method of control is being used on a number of other earth dams now under construction.

Let me now turn to the utilization of some modern developments in radiography and ultrasonics.

Radiography

Advances in nuclear physics and in X-ray techniques have given new tools which are finding a use both in the laboratory and the field. They are at present largely confined to research purposes, but the tools of research of one day often become the tools of control of a later day. Radiographic examination of metals started with X-rays and later the use of

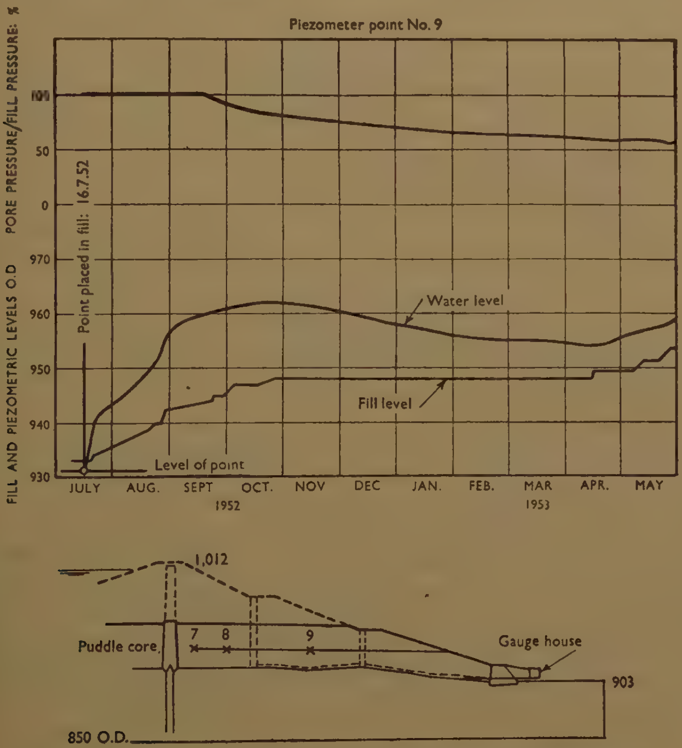

FIG. 9.—RECORD OF PORE-WATER PRESSURE IN AN EARTH DAM

gamma rays developed. X-rays have been less used as an engineering tool in the study of concrete, but one interesting application is in the study of the bond between reinforcing bars and concrete. Bond is a surface phenomenon and the fixing of a strain gauge to the steel interferes with the very object of the study. This difficulty has been got over in the past by the use of tubular reinforcement with optical or resistance-wire gauges fixed on the inside to measure the stress distribution during pull-out tests. In the radiographic methods now being developed, for example, by Professor Evans at Leeds, lead markers are fixed in niches in the reinforcing bar and their relative displacement is measured by X-ray photography. These photographs also show the slip between the bar and the concrete

as shearing action takes place between them. Even this method cann entirely eliminate interference with the bond, but it appears to reduce to a minimum.

A more fundamental use of X-rays, of interest at present primarily t the chemist and physicist, lies in the study of the atomic structure of th compounds present in fresh and hardened cement. But the object of th study, a proper understanding of the nature of the cementing action and c such basic properties as shrinkage and strength of concretes, is of muc importance to the engineer. Already we have important clues to at leas one of the causes of shrinkage of set cement. The ultimate crystal unit of the prime cementing compound of Portland cement, the hydrated calciu silicate, consist of sub-microscopic fibrous crystals with, on the atomi scale, a layer structure parallel to their length. Water molecules ar held as interstitial sheets between these layers to an extent that varies wit the state of dryness. As water is removed the layers come closer togethe and the crystal shrinks at right angles to its length, and when water i taken up the distance between the layers increases again and the crysta expands. There is, in fact, a close parallelism to the shrinkage and swellin of some clay minerals. This is probably not the only cause of dryin shrinkage in set cement, but in certain ranges of water content it may b the predominant one.

With the development of the electron microscope, using a beam o electrons in place of light, we can " see " particles much too small to b visible in the normal optical microscope. A picture of set cement is show in Fig. 10 (facing p. 257) at a magnification of about 20,000 diameters The felted lath-like crystals appear to be responsible for the strengt development since other hydrated calcium silicate crystals of closel similar composition are known which do not form this kind of structur and which have no cementing properties. The way to apply this know-ledge is not yet in sight, but we well know how increasing knowledge o the fundamental structure of materials eventually becomes a guide t future developments.

Let me return now to matters of more immediate practical interest t the engineer, such as the use of radiographic methods for the examinatio of materials, such as in-situ concrete. Here X-ray techniques are rarel applicable, since for a mass of any substantial thickness the equipmen required would be very costly and not suited to use in the field.

As a substitute we can use gamma rays. The gamma radiation emitte from radioactive elements is a form of hard X-rays, of smaller wave lengt and greater penetrative power. The extent to which they penetrate a material is dependent on its density and its chemical composition and o the energy of the rays. It so happens that throughout a fairly broad ban of energies, the absorption of gamma rays is little affected by chemica composition as far as the lighter elements, which make up materials suc as concrete and soils, are concerned. We have, therefore, a method by

which the density of concrete, or of soils, can be measured in situ. The principle of the method is quite simple. A source of gamma rays is placed on one side of, say, concrete, and the intensity of the incident beam, and of the beam that emerges on the other side, is measured. The Geiger counter, used so much in atomic physics, provides the measuring instrument. Laboratory tests on concretes of known density give the necessary data on the attenuation of the rays so that from the measured attenuation and the known path-length the density of an in-situ concrete can be derived. The method can be applied not only to hardened concrete, but also to concrete whilst still in the plastic condition, in contrast, for example, to the ultrasonic methods which cannot be used in concrete before hardening has started. A convenient source of gamma rays is now available in radioactive cobalt—Co^{60}—with which measurements can be made through concrete up to 3 ft thick. Penetration of thicker masses is possible, but it requires sources of too great a power to be practicable to handle in the field from the point of view of protection of the operator. Since variation of the density of a particular type of concrete bears a general relation to uniformity of compaction and to strength, we have here a method of non-destructive test which has been applied, for instance, by the Road Research Laboratory to the study of concrete road slabs. A further application is the use of gamma rays to locate the position of reinforcing bars in concrete members.

Another type of atomic particle, the neutron, enables a measurement of water content to be made. The neutron, that is, the particle of zero charge and of mass equivalent to the nucleus of the hydrogen atom, is scattered by collision with atoms on which it impinges and, as a result of this collision, the neutron loses part of its kinetic energy and is slowed down. This energy loss is much greater in collisions with atoms of very low atomic weight than in collisions involving heavier elements and, in the process, what are known as "fast" neutrons are converted to "slow" neutrons. In the case of most constructional materials the only element of very low atomic weight present in appreciable quantity is hydrogen in water molecules. If, therefore, a device for measuring slow neutrons is inserted into a material together with a neighbouring source of fast neutrons, and the rate of production of slow neutrons is measured, an estimate can be obtained of the moisture content. So far the method has mainly been applied to the measurement of the water content of soils in situ, but its application to other materials, such as concrete, is now being studied and may well be of greater importance. There are many purposes for which a knowledge of the moisture content of a material in a structure would be valuable but only destructive or cumbersome methods have previously been available. The neutron method is still at a relatively early stage of development, but we may undoubtedly look forward to improvements that will give a new tool of significance to the engineer.

Ultrasonic methods

The theory of the propagation of waves in solids goes back to the grea classical physicists of the last century—names such as Poisson, Rayleig and Kelvin—but, for long, its application was confined to seismic wave and to vibrations at audible frequencies. Its application to ordinar engineering measurement had to await, as we have seen, the developmen of means of producing and detecting high-frequency waves in solid These methods we now possess, thanks to the striking developments c electronic techniques.

In the control of the quality of concrete the engineer has largely bee limited to the tests that can be applied to the separate materials, or t samples of the concrete taken before placing. The quality of in-sit concrete has to be inferred from the control tests, and the supervisio exercised during placing. We now have a prospect that the measuremen of the velocity of propagation of ultrasonic waves will allow of an estimatio to be made of the quality of in-situ concrete. This method has numerou potential applications, though also certain definite limitations.

Much work has also been done, particularly by the Road Researc Laboratory in Britain and by laboratories in other countries, on th development and application of the method to the general study of concret and other engineering materials. It forms a valuable laboratory too though in this respect only as an alternative to other methods, but for fiel investigation on in-situ concrete it can provide data obtainable in no othe way short of taking cores.

The relation between the velocity at which ultrasonic waves are propa gated in concrete and its strength is not independent of the concrete mix o type of aggregate so that the method is a comparative, and not an absolut one, but the difference in wave velocity between well and badly compacte concrete is large, so that weak areas can be readily detected. Muc interest is now being shown in the development of the method as a mor quantitative tool for the estimation of the development of strength c concrete after placing in position. When, as in placing new concret check tests can be made on both the wave velocity and strength of speci mens prepared from a concrete, many of the sources of variation in th wave velocity/strength relation can be eliminated. We are here perhap only at the early stages of development of a tool which may eventuall find wide application in the control of concrete construction. The metho has also been applied, by a reflexion technique, to the measurement of th thickness of concrete road slabs.

Another important, and indeed the original, application of the methoc is to the survey of existing concrete structures. Since in such cases n samples of the original materials or concrete are available, the results hav to be interpreted with care. The pioneers in this method of surveying ol structures were the Hydro-Electric Power Commission of Ontario, wh have long maintained a substantial research laboratory to aid their worl

The apparatus developed there—called the Soniscope—has permitted measurements to be made on concrete dams and other structures up to 50 ft thick and has been found useful for the detection of weak areas of concrete, of internal cracks and voids, and of the depth to which surface cracks penetrate. By repetition of the measurements at intervals of, say, a few years, it should be possible to follow any progressive deterioration in a concrete structure.

The ultrasonic method has also been used in the laboratory for the measurement of the difference between the modulus of elasticity of clays at right angles and parallel to the laminations. It also has potential applications to the measurement of the properties of clays in situ in ways rather similar to those I have described for concrete.

In this survey I have tried to show how advances in the experimental techniques of science have influenced civil engineering research. Though I have used a broad brush and touched lightly on many subjects, I hope I may have given you some picture of the large armoury of weapons now at the disposal of the engineer and some indication of how he is using them.

The examples I have mentioned will have illustrated how the engineer by the imaginative application of scientific principles has increased his powers to measure; and measurement, as Lord Kelvin used to say, is the first step to real knowledge.

The development of science is unceasing; if it presents new problems to the engineer, it also brings new possibilities of solving them. When in 1828 Thomas Tredgold wrote his well-known definition of civil engineering he added :—" Its scope and utility will be increased with every discovery in philosophy and its resources with every invention in mechanical or chemical art." Those words still present a challenge to the engineer to seize new knowledge and from it to fashion new tools; a challenge to his imagination, that power of which Wordsworth speaks :—

> Imagination, which, in truth
> Is but another name for absolute power
> And clearest insight, amplitude of mind,
> And Reason in her most exalted mood.

Dr W. H. Glanville, in moving a vote of thanks to the Lecturer, described Dr Lea as an old friend and colleague; they had been together for about 30 years. In his opening remarks Dr Lea had spoken of turning from a family tradition of engineering to pure science, but Dr Lea's very practical Lecture indicated a strong engineering influence. They were all glad to see that.

Their thanks were all the more sincere because Dr Lea had been able to put the story forward in such a palatable form and to illustrate the points with so many interesting examples. The modern trend, of using large numbers of instruments, of which Dr Lea had spoken—others elsewhere had spoken of six hundred, when dealing with aeroplane structures—was terrifying. Dr Glanville said that when Dr Lea and he had begun their work, they thought they had enough to do with a dozen or so instruments, which had to be carefully chosen, with a good deal of imagination and as much insight as they could muster; the problem of sorting out so many observations presented many more difficulties than existed in the old days. No doubt Dr Lea spent a good deal of time, as did Dr Glanville, persuading the young men who worked for him that it was not just a game of collecting results and that the analysis of the results afterwards was important.

Dr Lea had said :—

> "If future development could give us a direct and reliable method of measuring stress in solids, as opposed to measuring strain, yet another most important weapon will have been added to the engineer's armoury."

How much had they prayed for such an instrument, such a method of measuring! Those who had had to measure movements in concrete, where there were temperature expansions, moisture expansions, creep, and elastic strain all coming into play had prayed for such a method of measurement, and he wished Dr Lea God Speed on his mission for the Holy Grail of the civil engineer. When it was found it would certainly solve the engineer's problems.

They had had a very interesting Lecture, which would add to the records of the Institution and which would be read with great pleasure. He very sincerely proposed the vote of thanks.

Mr H. J. B. Harding, seconding the vote of thanks, said he had known Dr Lea for 22 years, both professionally and, in recent years, as a member of the Building Research Board, and he was a great admirer of Dr Lea and his staff. At the periodic meetings of the Building Research Board it was borne home to one how much there was to know in the building field and how difficult it was to spread this knowledge. One of the notable points about the building research being conducted was that they were dealing with building materials which were expected to last for a very long time and yet manufacturers and others were extremely impatient if the Building Research Station could not give the complete answer to all problems immediately, with no time lag whatever! However, the Station did not let that disturb them!

Dr Lea had proved, as had Dr Glanville, that, with a scientific training

it was possible for what one might term an amateur member of the Institution to become a wise, capable, and active administrator, without a classical education. They were very fortunate in the Building Research Station to have such a live man leading such a live staff.

In concluding, he emphasized that only those who had prepared a lecture knew the hard work involved in it.

The vote of thanks was carried by acclamation.

ORDINARY MEETING

18 January, 1955

WILLIAM KELLY WALLACE, O.B.E., Vice-President in the Chair

The Council reported that they had recently transferred to the class of

Members

Abercromby, Edward George, B.Sc. (*Aberdeen*).
Aitken, Ian Miller Edington, B.Sc. (*Edinburgh*).
Bailey, Leonard Warner, B.Sc.(Eng.) (*London*).
Buckley, Alfred Henry.
Cave, Francis, Joseph, B.Sc. (*London*).
Cowan, Ronald John Primrose, B.Sc. (*Glasgow*).
Ely, Eric Harold, B.Sc.(Eng.) (*London*).
Glennie, John Forbes, B.Sc. (*Bristol*).
Holder, Egbert James Neville.
Hume, Harry Lancelot, B.E., B.Sc. (*New Zealand*).
Jenner, Henry Norman, M.B.E.
Lockett, Eric Barton.
McLean, Lachlan, B.Sc. (*Glasgow*).
Maylan, Arthur Leslie.
Scrutton, Harold.
Warrington, Eric George.
Wicks, Frank Sidney, B.Sc.(Eng.) (*London*).
Wolfe, James McMaster Bernard, B.A. (*Cantab.*).

and had admitted as

Graduates

Alaganandan, Sivasambo, B.Sc.(*Ceylon*).
Ball, Frederick Joe, B.Sc. (*Manchester*), Stud.I.C.E.
Ballantyne, Robert Hutchinson, B.Sc. (*Edinburgh*), Stud.I.C.E.
Bell, Allan Alick, B.Sc. (*Bristol*).
Best, Edward Faux, B.Sc. (*Nottingham*).
Bielous, Jaroslaw Wiktor, Stud.I.C.E.
Bill, James Ingram, B.Sc. (*Belfast*), Stud.I.C.E.
Bradford, Leslie William, B.A., B.A.I. (*Dublin*).
Brancher, David Marshall, Stud. I.C.E.
Brownlee, James Kerr.
Buckley, Kenneth Alfred, B.Sc.Tech. (*Manchester*), Stud.I.C.E.
Butler, Richard Edwin Vivian, M.A. (*Oxon.*).
Campbell, John Leslie, B.A., B.A.I. (*Dublin*).
Carlile, James Love, B.Sc.(Eng.) (*London*).
Chestnutt, Frederick James, B.Sc. (*Belfast*), Stud.I.C.E.
Chisholm, Ronald Arthur, B.Eng. (*Liverpool*), Stud.I.C.E.
Clark, Andrew, Stud.I.C.E.
Crossland, Roy, Stud.I.C.E.
Davies, David Oakley Mason, B.Sc. (*Wales*).
Davis, Michael Austen Robert, B.Sc. (*Bristol*).
de Soysa, Basil Mervyn, B.Sc. (*Ceylon*), Stud.I.C.E.
Douglas, Phillip Grahame, B.Eng. (*Sheffield*), Stud.I.C.E.
Douglas, Rodney Hugh Rovery, B.Sc. (Eng.) (*London*), Stud.I.C.E.
Downey, Eric, Stud.I.C.E.
Earwaker, Michael Charles, Stud. I.C.E.
Edwards, Douglas Charman, B.Sc. (*Birmingham*), Stud.I.C.E.
Emes, Peter Derek, Stud.I.C.E.
England, John Graham, B.Eng. (*Sheffield*).
Evans, David Royston, B.Sc. (*Wales*), Stud.I.C.E.
Fenton, Gordon Michael, Stud.I.C.E.
Foulsham, Roy Morton, Stud.I.C.E.
Fisher, John Hugh Bennett, B.E. (*New Zealand*).
Ganendra, Ponnusamy, B.Sc.(Eng.) (*London*).

Gemmell, James Duffy.
Gibbons, Michael Eddevane, B.Sc. (*Leeds*), Stud.I.C.E.
Gibson, George, Stud.I.C.E.
Graham, Peter John, B.Sc. (*Edinburgh*).
Grice, John Robert, B.Sc.Tech. (*Manchester*).
Haines, Lionel Peter.
Hall, Gordon Ward, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Hamilton, George Douglas, Stud. I.C.E.
Hamilton, John William.
Hannah, James, Stud.I.C.E.
Hawker, Geoffrey Fort, B.Sc.(Eng.) (*London*).
Hawks, Frederick William, B.Sc. (Eng.) (*London*), Stud.I.C.E.
Higher, Geoffrey Eric Gordon, B.Sc. (Eng.) (*London*), Stud.I.C.E.
Hill, John Raymond.
Howard, Michael Noel, B.E. (*National*).
Hudspith, John Clifford, Stud.I.C.E.
Hughes, Thomas Alexander, B.Sc. (*Belfast*), Stud.I.C.E.
Hultgren, David Edward, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Imbert, Ignatius Desmond Charles, B.E. (*National*).
Johns, Michael Charles Bedford, B.A. (*Cantab.*), Stud.I.C.E.
Khan, Nasir Mohammed.
King, Gordon William, Stud.I.C.E.
King, John David, B.Sc.(Eng.) (*London*).
Kirkland, Geoffrey Paul, Stud.I.C.E.
Klok, Cornelis.
Langley, Timothy John, B.C.E. (*Melbourne*).
Law, Thomas Henry, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Lewis, Kenneth, Stud.I.C.E.
Macdonald-Smith, Peter Graham.
McGregor, Graham Robert, B.Sc. (Eng.) (*London*).
McIntyre, John Alexander, Stud. I.C.E.
McKiernan, Bernard Ashbourne, B.E. (*National*), Stud.I.C.E.
McKillop, Colin, B.Sc. (*Glasgow*), Stud. I.C.E.
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The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Institution were accorded to the Authors.

Paper No. 6051

NEW NORTHAM BRIDGE, SOUTHAMPTON

by

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SYNOPSIS

Historical

The bridge forms the crossing of the River Itchen on the main Southampton–Portsmouth road, A3024, and was acquired from the Northam Bridge Roads Co. Ltd, by the Southampton Corporation in 1929 and freed from tolls.

The old wrought-iron bridge was constructed in 1889 on the same site as a timber bridge which had existed previously from 1796. From the date it was taken over by the Council, it was realized that the structure did not come up to modern standards and the Council were, in fact, ready to proceed with its reconstruction in 1939, but because of the outbreak of war, work was unable to proceed.

In 1945 the pre-war design was reconsidered and various consultations took place thereafter with the Ministry of Transport, culminating in a decision in 1948 to replace the old bridge by a prestressed concrete bridge.

Design

The design was controlled in varying degrees by the requirements of navigation during and after construction and by the need for accommodation of numerous services. The loading specified was the M.O.T. Loading Curve with a check for abnormal vehicles. Consideration was given to various types of deck and to the comparative merits of pre-tensioning and post-tensioning.

These various factors led to the adoption of a five-span bridge with a deck of precast pre-tensioned Tee-beams. The beams were connected together in situ, post-tensioned transversely, and made continuous over the piers to give finally a monolithic structure.

Construction

The main beams were precast on the site in pairs in specially constructed reinforced concrete pits, designed to absorb the reaction from the stressing forces. From the pits the beams were transported by light railway to a landing stage and lifted by a floating crane, which was warped into the river between the piers, where the beams were lowered directly into their final position.

The main piers, which are founded directly on gravel, were constructed inside cofferdams on a concrete foundation poured under water.

Introduction

Northam Bridge carries the main road (Ministry of Transport road number A3024) from Southampton to Portsmouth and the east across the

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River Itchen; its situation in relation to the town centre and to other main routes leading to the town and the site plan are shown in Figs 1a and 1b.

FIG. 1a.—KEY PLAN

The River Itchen flows approximately from north to south and forms a natural barrier between the east and west sides of the town. Bitterne and Woolston on the east side have a population of approximately 62,000 (about one-third of the total population of the town), and have grown up almost entirely since migration east of the river started in the late 19th century.

Apart from Northam Bridge there is a steam ferry service (the Itchen Ferry or Floating Bridge) about $1\frac{1}{2}$ mile downstream which connects Woolston with the lower end of the town. Although of considerable value to pedestrians and cyclists, the ferry is of little use to vehicular traffic and practically all such traffic makes the detour of 2 to 3 miles *via* Northam Bridge to and from the town.

A mile upstream from Northam Bridge there is another crossing at Cobden Bridge built originally by a private company of land owners and

FIG. 1b—SITE PLAN

dedicated to the public on completion in 1883. The Corporation in 1928 demolished the original bridge and replaced it by a reinforced concrete arch bridge of five spans, having a width between parapets of 45 feet. Because of weight restrictions on Northam Bridge, it is necessary for all cross-river traffic of gross laden weight exceeding 10½ tons to use Cobden Bridge. This bridge, however, is too far north of the town centre to be regarded as a satisfactory location for the main river crossing. It does not lie on any main route into the town and cannot be regarded as more than a convenient link between the adjacent districts on either bank of the river. Had this river crossing been in a more favourable topographical position, it is almost certain in these days of restriction of capital expenditure that it would not have been possible to carry out the new works described in this Paper.

From the foregoing it will be quite clear that, besides having to cater for through traffic, Northam Bridge carries the greater part—at least 75 per cent—of local traffic between the western and eastern parts of the town.

A twelve-hour traffic census taken on the 15th September, 1950 between 7.0 a.m. and 7.0 p.m. gave the results in Table 1.

TABLE 1

	Cars	Public service vehicles	Commercial vehicles	Motor cycles	Pedal cycles
Traffic to town	2,916	624	1,443	551	2,216
Traffic from town	3,031	668	1,880	559	1,578
Total traffic for 12 hours	5,947	1,292	3,323	1,110	3,794
Average hourly flow	496	108	277	92	316
Peak hourly flow	1,002	195	333	258	597

HISTORICAL

On the left bank of the river on the northern approach to the bridge i the site of the old Roman settlement of Clausentum, and it is highl probable that in those days there was some form of communication acros the river. It is certain, however, that after Roman times and until th end of the 18th century, there was no bridge across the River Itchen a this point.

The known history of this crossing commences in 1796 with an Act o Parliament setting up " The Company of the Proprietors of Northa Bridge and Road " and conferring statutory powers for the building of

bridge and certain roads on that company. The preamble of the Act, which gives a good general picture of the lack of communication between Southampton and the east, reads as follows :—

"Whereas there is at present no direct communication between Southampton, the several parishes and places contiguous thereto, lying on the eastern side of the River Itchen, Botley, Titchfield, Wickham, Gosport, the Town of Portsmouth and Portsea, Havant and Emsworth and the City of Chichester, and various other parts of the Counties of Southampton and Sussex, except by a ferry over the said River Itchen : and whereas from the violence of the winds and sea, the passage over the said River by the said ferry is often dangerous and frequently totally prevented, by means whereof any communication between the said places becomes impracticable and great inconvenience and loss is thereby occasioned : And whereas from the great increase of, and still increasing population and Trade of the Town of Southampton and its neighbourhood and from the vicinity thereof to and extensive intercourse between the several Places before mentioned, the building of a Bridge at or near Northam, within the Liberties of the Town and Country of the Town of Southampton over the said River Itchen to the opposite Shore at or near Bitterne Farm, in the Parish of South Stoneham, in the County of Southampton, and the making and opening of Public Roads to communicate with those Places, and with such bridge at each end thereof, is become an Undertaking of great public necessity, and will, in a very essential manner contribute not only to the Advantage, convenience and accommodation of the Inhabitants of the said town of Southampton and its neighbourhood, and the several other places before mentioned, and all persons travelling as well to and from the Places, as to and from various other parts of the counties of Dorset, Somerset, Wilts, Southampton and Sussex, but also to the public in general, by preventing and avoiding the Delays, Difficulties, Dangers and Losses which have been so frequently experienced from the want of some other passage over the said River, besides the said ferry."

The bridge subsequently erected by this company (Fig. 2, facing p. 280) was built entirely of timber and remained until 1889 when it was replaced by the old wrought-iron bridge. The timber bridge had five equal spans of 60 feet providing a clear waterway of 52 feet between each span and a width between parapets of 24 feet.

The wrought-iron bridge (Fig. 3, facing p. 280) was built in 1889 at a cost of approximately £9,000, and had three spans of 90 feet each, brick-built abutments, and twin cylindrical intermediate piers carried down to a suitable foundation. The width between the parapets is 24 feet, providing a carriageway width of 18 feet.

The Northam Bridge Company remained in existence until 1929 when the Southampton Corporation obtained parliamentary powers to acquire their assets and the bridge was actually handed over to the Corporation and freed from tolls on the 16th May, 1929.

It is rather remarkable that this bridge, which was designed to cope with horse-drawn traffic, and was, within a few years of being opened, having to carry much heavier and faster traffic owing to the advent of the internal combustion engine, has stood up for so long a period to the new conditions imposed on it.

The inadequacy of the bridge to meet modern traffic requirements was becoming apparent at the time the Council took over the bridge and this fact, coupled with the considerable amount of deterioration in the structure which had occurred since it was built, led the Southampton Corporation to consider reconstruction proposals. During 1935–1936 the then Borough Engineer and Surveyor of Southampton reported to the Council in some detail on various alternative schemes for the reconstruction of the bridge. One of the alternatives, which at the time appears to have been the subject of a fairly full investigation, is of interest since it proposed to replace the bridge by a weir lock and a short-span bridge, and thus form an inland basin for yachts and small craft upstream of Northam Bridge. One of the major reasons which led to the scheme being turned down was that the basin so produced would have created drainage difficulties in those parts of the town which had sewer outfalls into the River Itchen upstream of Northam Bridge. The Council finally decided on the construction of a three-arch bridge in reinforced concrete on the site of the existing bridge, having an overall width of 64 feet between parapets, and a contract for its construction was, in fact, let in the summer of 1939. The contractors had moved in some of their equipment by the time hostilities broke out in September 1939, but the contract was determined since work had not actually begun.

During the 1939–45 war the wrought-iron bridge sustained considerable damage from bomb blast and splinters and from near misses, and because of this and the further general deterioration of the structure during the war years, it was found necessary to provide additional supports by means of steel grillages erected on piled-timber trestles in order to avoid the imposition of any further weight or speed restrictions, the bridge already having been subject for many years to a weight restriction of 10½ tons gross, and a speed limit of 15 miles per hour. These works were carried out by Messrs Holloway Bros at a cost of £14,000.

The question of the location and design of the proposed post-war bridge has gone through several stages. During the war years a proposal was put forward to build an entirely new bridge at Millstone Point, about half-a-mile downstream from the old wrought-iron bridge but, because of the expense of linking a bridge at this point with the town's existing road system, it was decided once and for all that the new Northam Bridge must be on, or near, the site of the old bridge.

At the end of the war in 1945 the Borough Engineer re-examined the pre-war design for Northam Bridge, and after consultation with the Ministry of Transport, it was decided that this design should be revised.

It has already been mentioned that the 1939 proposal was for the new bridge to be erected on the line of the then existing structure, and a temporary bridge was to be erected downstream to carry traffic, but when the scheme was reconsidered, it became obvious that the level crossing on the Northam approach would also have to be eliminated ; this had not been provided for in the pre-war scheme. To enable the railway diversion to be undertaken, and to improve the re-alignment of the Bitterne approach as well as to facilitate the work of construction, it was decided that the new bridge should be erected just far enough upstream from the old bridge to enable pile driving, excavation, and construction works to be carried out entirely clear of the old bridge and at such a distance as not to endanger its foundations during the construction period. The Borough Engineer had reason to believe that if the old wrought-iron bridge was to be maintained in a serviceable condition until the new one was completed, any interference with its foundations should be avoided so far as possible.

In 1948 the Council put forward to the Ministry of Transport a scheme for a reinforced concrete bridge of five spans of similar profile to the prestressed concrete bridge which has now been erected, but was informed by the Chief Engineer of the Ministry at that time that, although his Department fully recognized the immediate necessity for a new Northam Bridge, it was not then possible to find either the steel or the cement for a bridge of such size and, in order to achieve economies in both these scarce materials, it was suggested that the question of the use of prestressed concrete in the structure should be investigated fully.

With this in view Messrs Rendel, Palmer & Tritton were asked to submit a preliminary report on the probable cost of the bridge redesigned as a prestressed structure. Their report was accepted by the Ministry of Transport, and work then went ahead on the preparation of the detailed scheme. Tenders were invited in August 1951, the lowest received being that of Messrs Christiani & Neilsen Ltd, and work was commenced on the 1st April, 1952, the official starting date awarded by the Ministry of Works.

Design

Bridge arrangement

Before preparing the report already referred to, the consulting engineers carefully examined the site and the soil data available.

The alignment had previously been fixed and it was necessary to determine the length of suspended structure in the light of the desire to use earth embankments to the greatest practicable extent to save steel. It was seen that banking-out to the line of the abutments of the old wrought-iron bridge would result in a maximum height of filling of 35 feet and would

require the forming of high banks on the foreshore which, in places, slopes fairly steeply towards the low-water channel. In view of the cost of such work and the difficulty of stabilization, it was decided that a bridge length of about 500 feet would be necessary, extending from the river bank on the south side to the half-tide flats on the north.

The profile is controlled by the head room over the diverted railway on the south bank and the navigation clearance beneath the central portion of the bridge, which was fixed by the Harbour Board at 15 feet. In meeting these requirements it is, of course, desirable to keep the road level as low as practicable to reduce gradients and the quantity of embankment filling. This might have been realized by adopting numerous short spans but it was found that the accommodation of the various services, comprising two 18-inch and two 9-inch gas mains, two 10-inch water mains, twenty-four 4-inch telephone ducts, and eight electricity cables, set a limit to reduction of construction depth and this, in conjunction with the high cost of pier works in the tidal river and also the necessity of keeping the new piers clear of the navigation channel running between the auxiliary trestle supports of the old bridge, led to the adoption of the arrangement shown in Fig. 6, Plate 1, consisting of five spans, the three centre-spans being 105 feet and the two side 85 feet. The completed bridge is shown in Fig. 15 (facing p. 281).

The width between parapets is 65 feet to accommodate two 22-foot carriageways, a 5-foot central reserve, and two 8-foot footpaths.

Piers

The soil on the site of the new bridge consists of layers of mud and peat overlying a stratum of coarse ballast about 8 feet thick; below is sand generally fine, extending down to a considerable depth.

At the time when the basic drawings were prepared it was essential to minimize the use of steel even for temporary works. Accordingly, the pier foundations were designed to be carried on pre-tensioned prestressed concrete piles driven into the fine sand with the heads finishing at about high water level; spaces between the piles were to be infilled with grouted aggregate up to low water level and with concrete placed in the dry above, tidally where necessary; scour was to be countered by chemically consolidating the "block" of ballast between and around the piles. Thus, the use of a steel sheet-piled cofferdam would be avoided.

The restrictions on expenditure and the various controls in operation led to a lapse of several years before the contract was let. Meanwhile steel sheet-piling had again become available to a limited degree and the contractor proposed the use of a cofferdam; this was agreed. On examination of details it was seen that a small saving in cost would be realized by increasing the widths of the pier bases beyond that originally proposed since it would then be feasible to eliminate the bearing piles and found

directly on the ballast, and by re-arranging the chemical consolidation in the form of a curtain against scour to permit the sheet-piling being recovered for re-use. The form of foundation finally adopted is shown in Fig. 4, Plate 1 and Fig. 5; the construction is described later in the Paper.

The pier shafts are of normally reinforced concrete construction.

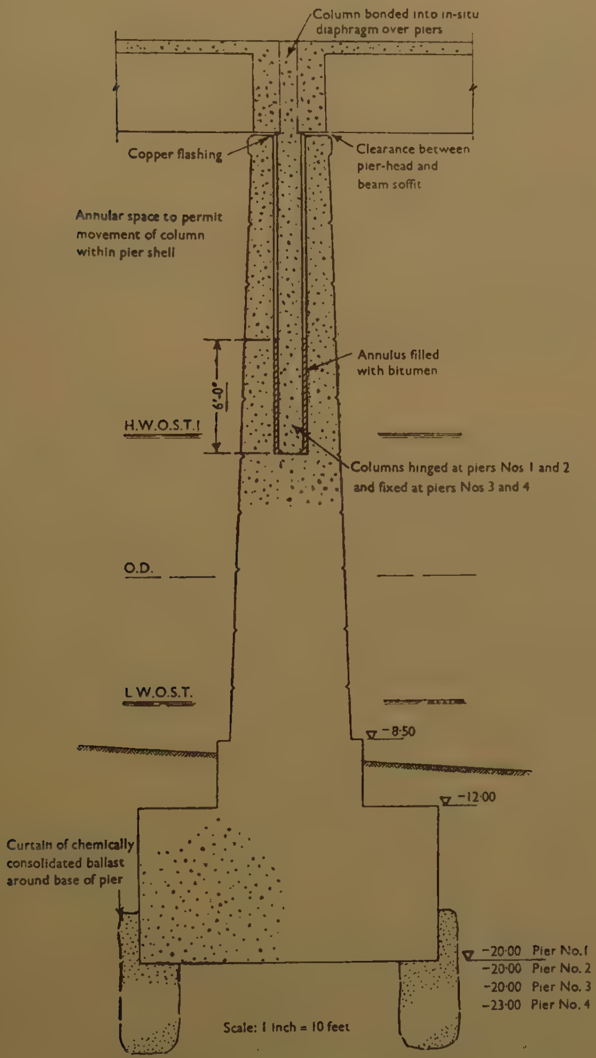


FIG. 5.—CROSS-SECTION OF PIER

Columns for the support of the bridge deck, separated by an annular ga from the body of the piers, are formed in the upper portions ; these ar further described in the section dealing with the deck. The steel reinforce ment to the shafts is comparatively light and has no feature worthy o particular mention.

Abutments

The abutments are founded on vertical and raking prestressed concret piles.

The north abutment is of box form with flying wing walls extendin back to assist in holding the embankment filling. In addition to it functions as a retaining wall and bridge support, it is designed to provid an anchorage to the superstructure.

The south abutment above pile-cap level consists of two comparativel thin and flexible concrete walls carrying the south end of the main bridg and the north end of the railway bridge respectively. To give an appear ance of greater solidity, a screen wall is provided to close the space betwee the bearing walls at the ends, small clearances being arranged to permi expansion flexure of the bearing members.

Deck

General

The bridge was required to be designed to carry Ministry of Transpor Standard Loading plus Abnormal Loading Class C.

The deck (see Fig. 8, Plate 2) consists of prestressed concrete in th form of pretensioned precast beams, sixteen per span, which were, afte erection, connected one to the other by concrete placed in situ, the dec being then stressed transversely by post-tensioned cables.

The parapet beams are, as precast, of rectangular cross-section, whils the inner ones are Tee-beams except for a length at the ends adjacent t the piers where the flanges are narrowed to give a substantially rectangula section.

Continuity

The beams act as simply supported members in respect of their ow weight and that of the small amount of in-situ structural concrete, bu for added dead loads and live loads the bridge becomes a five-span con tinuous structure (see Fig. 7, Plate 1).

Continuity over the piers is effected as regards top tension by mean of pre-tensioned precast junction slabs 35 feet long centred on the pier and placed in the spaces provided by the narrowing of the beam flange The sides of these slabs are connected to the beam stems, in which th wires are bent up as shown in Fig. 8, Plate 2 to give the correct arrange ment of prestresses, and the assembly is prestressed transversely as fo

the beams. The ends of the precast beams are embraced by a transverse diaphragm over each support which serves to transfer the reaction and, in the case of the piers, to transmit the bottom compression arising from the continuity moment.

The minimum length of the junction slabs and their lap with the beam stems is determined by considerations of shear in the joint, or rather of the principal tension arising therefrom after diminution by prestressing. There is some similarity to a cover plate of a steel girder where the length is governed by the number of rivets required, plus an arbitrary allowance. Actually, it was found that the permissible point of curtailment of the beam flanges was at such a distance from the piers that no difficulty arose in providing a generous lap.

Expansion

As indicated above, the bridge deck is anchored at the north abutment and is supported on columns at the piers and on a wall at the south abutment. The anchorage forms a nodal point in relation to length changes arising from expansion, contraction, and shrinkage which, of course, increase progressively to reach a maximum at the south abutment. To cater for these without the introduction of mechanical devices the columns and wall are made flexible.

The columns are placed on the lines of the beams and are connected to them through the embracing transverse diaphragm with which they make a fixed-end joint. They are fairly heavily reinforced, but the load carried is such that the necessary scantlings tend to militate against flexibility so that, whilst at the two northernmost piers the smaller movement permits of the bases also being fixed, rudimentary hinges are necessary at the bottoms of columns on the remaining two piers to avoid tensile stresses. The wall support at the south abutment, being of the full width of the bridge and subject to a smaller reaction, requires but small thickness, and so adequate flexibility is obtained notwithstanding the movement at this point being a maximum.

The annular clearance gap surrounding the pier columns is filled with bitumen to a point above high water level, thus affording additional protection to a member not readily inspected or maintained.

Transverse strength

In addition to the "solid" diaphragms at the supports which have a multiplicity of functions, "open" diaphragms are provided at about 14-foot spacings over the spans to augment the transverse strength of the deck and provide for the passage and support of the various service pipes. These intermediate diaphragms are of contained-triangulation form and the bottom chord is prestressed by means of two 12-wire Freyssinet cables which give adequate strength to permit outer beams sharing in the support of heavy loads.

Prestressing

Early consideration was given to alternative construction schemes.

The possibility of wholly-in-situ placing of the concrete was examined and it was found that great difficulty would arise in the provision of timber for the requisite extensive staging and in preserving the necessary channel and clearance for navigation. Little was known (in 1948) about friction losses with long curved cables in cored holes which might or might not be truly formed and accurately placed, and it was feared that the losses might be such as to defeat the design. (Subsequent events and research have indicated that the fear was well founded.) These factors taken together led to the decision that the main beams should be precast, in span lengths, in a suitable yard on shore, transported, erected on the piers, made continuous and monolithic, and transversely prestressed using comparatively short straight cables.

Precasting having been decided upon, consideration was directed to the question of pre-tensioning or post-tensioning, the latter method not being open to the same objection as with in-situ work since only span lengths would be cast and supervision could be more intensively applied. Designs were roughed out and " paper " solutions obtained for the two methods, which were then compared. It was seen that pre-tensioning would require a stressing bed which would represent appreciable cost and special bend anchors would have to be provided. Post-tensioning would call for a very large number of cores which could not be accommodated in the scantlings as determined by stress considerations alone ; thus, additional weight would be involved. Furthermore, curved or bent cables would be unavoidable, fixing and maintaining the numerous cores in position during concreting promised to be a major problem and full effectiveness of the grouting would be doubtful. Comparing the cost aspects of the relative disadvantages, it was found that, in view of the repeated uses possible, the stressing bed would not represent an excessive charge per beam and after allowing for the bend anchors it was estimated that the on-cost involved in pre-tensioning was actually rather less than that resulting from the increased concrete and the coring, fixing, and grouting of the post-tensioning scheme. The main factor influencing the decision in favour of pre-tensioning was, however, its relative freedom from fear of the unknown. More recently, improved methods and greater experience have considerably reduced the uncertainties in post-tensioning, but the opinion is expressed that their weight in the balance continues to be of some significance.

All wire in the bridge deck is 0·200-inch diameter of 100 – 110-tons-per-square-inch ultimate strength. It was required to be pulled up initially to an effective stress, after allowance for jack and guide losses, of 74 tons per square inch reducing after an interval of 2 minutes to 70 tons per square inch at which stress it was anchored. Losses arising from compression, shrinkage, and creep of the concrete, and creep of the steel were assumed to be 15 tons per square inch. Observations taken indicate

FIG. 2.—THE OLD WOODEN NORTHAM BRIDGE

FIG. 3.—THE WROUGHT-IRON BRIDGE

Fig. 11.—Stressing pit laid out for junction slabs on left and beams on right

Fig. 12.—Bulkhead in stressing pit showing different types of anchorages

FIG. 13.—FLOATING CRANE FOR 105-FOOT BEAM

FIG. 14.—85-FOOT BEAM AND TRAVERSER CARS

Fig. 15.—View of completed bridge

this allowance to be generous for the highest grade of concrete. A certain amount of end-slip of the wires on release was anticipated, but none was, in fact, measurable except in the case of a few early beams where some end-splitting of the concrete was observed. Such splitting was overcome by the swelling out in the form of a hammer head of the portion of the beam stem to be encased in the diaphragm after erection.

Concrete

Concrete for prestressed work was specified to be not leaner than a nominal $1 : 1\frac{1}{2} : 3$. Grading and the minimum water/cement ratio consistent with thorough compaction in the work by vibration were required to be found by trial, with the limiting proviso that the water/cement ratio should not in any case exceed 0·40 by weight. The use of rapid-hardening cement was permissible.

After numerous trials the mix proportion was fixed at 1 : 3·80 by weight with $\frac{3}{4}$-inch-maximum-size aggregate and the water/cement ratio at 0·30. With such concrete normal-hardening Portland cement was found to give a sufficiently high early strength and its use for the great majority of the beams eliminated shrinkage troubles encountered on the earlier ones, for which rapid-hardening cement was employed.

The cube strengths specified were, at release, 5,000 lb. per square inch and at 28 days, 7,000 lb. per square inch. No difficulty was experienced in obtaining the necessary strength.

Testing

The specification called for certain non-destructive testing of various members to induce a tensile stress in the concrete of 300 lb. per square inch without cracking; the requirements were complied with, and there was no cause for concern as a result of such testing.

In the case of one of the first pair of beams made, which were parapet beams, there was appreciable shrinkage-cracking before release and this beam suffered, by mischance, considerable distortion and jolting during removal from the stressing bed. All cracks resulting therefrom closed up so as to be quite invisible, but it seemed that this might be the weakest beam in the bridge and it was decided to test it to destruction since this appeared to offer a worthwhile opportunity of confirming the basis of the detail design.

The test beam was 105 feet long and it was tested on a span of 101 feet 6 inches by means of jacks thrusting against overhead transverse joists which were anchored down to concrete blocks let into the ground, the loading being applied symmetrically about the centre at points 15 feet apart.

A deflexion of 1 inch was recorded at the first crack, which occurred at a bending moment of a little more than half the ultimate moment. As loading continued to the maximum extensive cracking developed and the

deflexion increased to 6 inches. Without further loading the deflexion continued to increase, reaching 7 inches after a period of 8 minutes when a horizontal crack appeared about 3 inches below the top of the beam. During the next 2 minutes further deflexion and widening of cracks was observed before the beam finally failed.

No wires were broken and failure was by crushing of the concrete at a calculated stress of 8,500 lb. per square inch, the corresponding steel stress being 95 tons per square inch. It should be noted that notwithstanding failure by crushing the collapse was not sudden, taking about 10 minutes in all.

It should be further mentioned that, although as tested the beam was "over-reinforced," in the finished work the parapet beams are made composite beams by the addition of an in-situ concrete plinth capping and slab connecting to the adjacent beam, and they then become "under-reinforced."

The load to be carried by the parapet beams is not closely determinable since, in addition to that directly imposed, they share in the support of the carriageway live load. It was estimated, however, from considerations of longitudinal and transverse stiffness that the maximum working bending moment would be about 80 per cent of that of the inner beams and on this basis, after allowance for the incomplete nature of the beam as tested, it is reckoned that the cracking and failing moments represent excesses of not less than 45 per cent and 170 per cent respectively; these are considered to be satisfactory.

Ancillary bridges

The small bridge over the diverted railway immediately to the south of the main bridge consists of a normally reinforced concrete slab deck spanning from a bearing wall on the main abutment pile cap to a piled buttressed wall retaining the filling to the south approach embankment. To obtain the required headroom it was necessary to pass the gas mains through sleeves built into the thickness of the deck slab. It was fortunately found possible to carry the water mains at a low level, under the rail track, up to the main bridge.

A 33-foot-span bridge is provided in the north embankment in line with a span in the approach to the former bridge to preserve a channel for the passage of water, which was necessary to ensure the scouring of silt from the timber ponds upstream. The abutments are of reinforced concrete construction carried on prestressed concrete piles and take the form of propped retaining walls. The deck is of composite construction consisting of pre-tensioned precast inverted Tee-beams with a filling and covering of in-situ reinforced concrete; suitable provision is made to afford horizontal support to the top of the abutment walls.

Approaches

The approaches generally are of the solid embankment type (see Fig. 9, Plate 2), the filling consisting of a loamy ballast with good compacting qualities.

In the case of the section of the north embankment which had to be formed over the tidal flats, it was desired, if possible, so to place the filling that the soft material, consisting of mud and peat, was rolled away from the advancing toe of the bank to ensure founding on the natural ballast stratum below. To assist such gravity displacement, employment of explosives in the toe-shooting manner was contemplated, and a provisional sum was set aside for this purpose. As described in the section dealing with construction, gravity displacement worked well initially but could not be kept going once a mud roll had built up in front of the toe.

Unfortunately, the scheme envisaged during the design stages for toe-shooting could not be applied owing to local objections to the necessary scale of the work and the lack of facilities for importing a suitable slow-burning explosive. The two trials which were made were in the nature of a forlorn attempt to avoid the costly alternative of dredging.

Finishes

The carriageway bottoming on the embankments consists of hand-pitched stone blinded and rolled to a finished thickness of 10 inches.

The road surface is of rolled asphalt 2 inches thick on the main bridge and the railway bridge and 4 inches thick elsewhere. Kerbs are of Swedish granite and footway pavings of precast concrete flags. The parapet railing is of zinc-sprayed mild steel with a handrail of bronze-covered hardwood ; lamp standards consist of cast-iron columns with welded mild-steel bracket arms, the whole being zinc-sprayed.

Embankment slopes below extreme high water are protected by stone pitching ; above they are soiled and grassed. To prevent erosion before the roots got a good hold, the grassing was carried out by laying intersecting diagonal strips of turf to form an open trellis pattern, the diamond-shaped spaces being soiled and sown.

Construction

Site arrangement

The only ground available as a working space at the commencement of the contract was a children's recreation ground immediately west of the Plaza cinema. This site, approximately 300 feet by 100 feet, was used for the construction of the main prestressed units and for offices, fitters' shop, blacksmith, etc.

The area of the future south embankment was occupied by an iron foundry, various buildings, and a car park for the cinema. Filling to the embankment in this area commenced immediately upon demolition of the existing buildings.

A small triangular area west of the south approach became available after in-filling over the tidal flats for the diversion of the railway sidings, and this area was used for the storage of prestressed units and reinforcement. The contractor's general plant and equipment yard was situated immediately east of the old bridge approach.

On the north side of the river the site of the new embankment was, at the commencement, occupied by stored timber and a timber shed, which had to be demolished and re-erected. This ground also was turned over to permanent construction as soon as it became available.

The existing branch-railway which separated the casting yard from the remainder of the site had to be kept open to traffic and was spanned by a removable bridge carrying a 3-foot-gauge railway track for the transport of beams.

These rather restricted working sites, together with the necessity for maintaining free navigation in the river at all times, and for leaving the branch railway open to traffic, when required, decided the construction layout and certain methods used.

A mixing plant consisting of a 10/7 pan mixer and a small weigh-batcher was erected in the casting yard for the construction of precast units only.

The concreting plant for the river piers consisted of a $\frac{1}{2}$-cubic-yard mixer and a weighing platform for the aggregates and cement, erected on a barge which also carried sufficient aggregates for a day's output. The barge was loaded by conveyor belts at a landing stage ashore. All concrete was placed from bottom-opening skips by a mobile crane travelling on the staging surrounding the piers. This concreting plant was later moved ashore for the construction of the north abutment and the scour span foundations.

Bulk cement in 3-ton portable containers was used in both cases

Beam construction

The maximum forces to be resisted in the pre-tensioning of the main reinforcement in the heaviest beam were 550 tons horizontal reaction and twenty-two near-vertical reactions of approximately $10\frac{1}{2}$ tons each at the positions of the bend anchors. In addition, 30 tons downward thrust occurred at the end shutter of the beam where inclined wires passed through

Methods of construction were studied, incorporating independent anchor blocks and heavy steel shuttering, but these were discarded in favour of a reinforced concrete pit (Fig. 10, Plate 2) with comparatively thin walls and bottom slab (Fig. 11) designed to withstand the stressin

forces and giving sufficient working room round the beams for easy erection of the shuttering. The horizontal force of 550 tons was transferred to the pit walls by heavy steel bulkheads. The vertical downward-acting forces of $10\frac{1}{2}$ tons each were taken by R.S.J's spanning the pit, and the vertically upward-acting forces by fixing the bend anchors to the pit bottom. The anchor bolts holding the bend anchors were hinged at the point of fixing, in order to make the bend anchor move with the wire during stressing, thus avoiding friction losses and the danger of breakages from pulling the wire through a sharp bend. At the end shutter, where some of the bent-up wires changed direction, they were passed over steel rollers, giving minimum friction.

So far as reinforcement was concerned, the eighty beams in the structure consisted of four types, which were either identical or symmetrical in pairs; it was, therefore, decided to stress two beams in series and to construct the pit about 240 feet long. The pre-tensioned junction slabs were constructed on the long-line principle in two tiers of six slabs each inside the pits.

One pit was equipped for 105-foot beams and junction slabs, the other for both 105- and 85-foot beams and junction slabs. In this pit a single beam could also be made if necessary.

The wires were pulled through the pits singly by hand and passed through the bend anchors, the holes in the end shuttering and the bulkhead, and were stressed in groups of eight by means of Freyssinet jacks; eight wires passed through each bend anchor. Stressing was carried out simultaneously at both ends of the pit, maintaining equal elongation by means of a telephone connexion, thus ensuring that the wires remained stationary in the centre of the pit between the ends of the two beams.

All the wires in a pair of beams were pulled hand-tight after fixing the bend anchors at predetermined positions to allow for elongation. The wire groups were then stressed, working from the bottom towards the top, thus avoiding the moving bend anchors fouling wires already stressed. End anchoring of the wires was first attempted by the use of Freyssinet cones; it was thought that these could be used a reasonable number of times. Experience proved, however, that after one or two uses the wires would slip, owing to wear on the female cone, and the anchoring was, therefore, altered to the use of spring-loaded grips anchoring each wire separately. The eight wires passing through each $1\frac{1}{2}$-inch-diameter hole in the bulkhead plate were spread out through eight holes on a circle in the bottom of a steel cup. A further steel cup with eight holes, on which the stressing jack rested, was then threaded on to the wires after threading on the eight steel grips. The depth of this latter cup was made the length of the anchor grip plus the estimated elongation of the wire for a stress of 4 tons per square inch, which was the overstress required before anchoring. When releasing the jack the grip would consequently automatically engage at the correct stress of 70 tons per square inch. Fig. 12 shows single-grip

anchorages in the upper portion and Freyssinet cone anchorages in the lower portion. The Freyssinet cones were cast into a concrete block.

The release of stress was accomplished by means of four 150-ton hydraulic jacks at one end of each pit. The jacks pushed the bulkhead out sufficiently to remove a 2-inch steel packing and were then slowly let back. Before this operation was carried out it was necessary to remove all bend-anchor bolts and, owing to the internal moment in the beam, it was further necessary to hold down the beam in four places so that the natural hogging of approximately 1 inch could be allowed for by gradual release of both the horizontal stress and the R.S.J's placed across the top.

Before deciding upon the final method of construction a full-scale replica of a group of eight wires was stressed on a specially constructed test-bed and the strains were measured electrically in order to ascertain the feasibility of stressing two beams in series and to measure the strain losses which would occur at the points of change of direction of the wires, whether in a bend anchor or on an end roller. It was found that the strain losses were at no point more than 8½ per cent; the average of four points along the length was 6 per cent.

Shuttering to the beams, as elsewhere in the structure, consisted of ¾-inch plywood made into standard panels and strutted to the sides of the pit. The plywood was screwed to timber battens and the countersunk screw-holes were filled in with putty.

The placing of concrete in the beams was considerably impeded by the close spacing of wires and, after several experiments on short stretches of replicas of a beam, it was decided to use a rounded sea-borne aggregate with a maximum size of ¾-inch, the actual proportions being 1 cement : 1·07 sand : 0·66 ⅜-inch aggregate : 2·07 ¾-inch aggregate, by weight, with a water/cement ratio of 0·3. The concrete was vibrated by internal vibrators, wherever it was possible to pass these through the wires, aided by external electric vibrators, particularly around the bend-anchor positions.

The beams were released when the concrete had obtained a strength of 5,000 lb. per square inch, which, during the summer period, usually occurred 3 days after concreting.

Construction of prestressed units in the pits, consisting of eighty beams and sixty slabs, commenced on the 1st November, 1952, and the last unit was lifted on the 4th April, 1954—in all 74 weeks, including holidays. The average turn-round for a pair of beams was 18 days and the minimum 12 days.

Beam handling

Immediately following the release of stress the beams, the heaviest of which weighed about 48 tons, were lifted from the pit by means of chain-blocks. They were placed on turn-table bogies and transported on a 3-

foot-gauge temporary railway to a stacking area or to the temporary landing pier, which was constructed immediately upstream of the south abutment. From here they were lifted by a floating crane, consisting of a pontoon with two shearlegs overhanging one side. Power used was a treble-drum steam winch, capable of lifting or lowering each drum independently or together. (Fig. 13, facing p, 281.)

The floating crane was warped into position between the piers by hand winches and the beam placed on $\frac{1}{2}$-inch-thick beechwood blocks on the pier top. After construction of the in-situ diaphragm over the pier the beechwood blocks were removed, thus transferring the load to the columns inside the piers and freeing the superstructure from the pier surrounds.

In the end spans, where the river-bed level did not allow entry of the floating crane, the beams were placed on traverser cars running on staging, pulled into position by hand winches, and lowered by hydraulic jacks. (Fig. 14, facing p. 281.)

Owing to the peculiar tidal conditions of Southampton Water, a period of 2 hours of slack water was available at high tide ; this proved to be ample for the operation.

Main river piers

. The river piers were constructed inside cofferdams of steel sheet-piling, driven by a McKiernan Terry hammer and a mobile crane travelling on temporary stagings. After closing the cofferdam the ground was grabbed to foundation level and the 7-foot-thick base poured underwater by tremie in compartments 14 feet wide by approximately 10 feet long. Division walls between compartments were formed by precast concrete slabs held in the indentations in the sheet-pile wall.

The natural river gravel was expected to be pervious for a considerable depth and since the weight of the underwater concrete base, together with the weight of the sheet-piles and strutting, was sufficient to resist only 20 feet of upward water pressure at the bottom of the base, out of the 26 feet which were possible at high tide, 6-inch-diameter tubes were cast into the underwater concrete and extended to high water level in order that the water pressure under the base could be observed at all times.

In the case of one of the piers, the pressure under the base slab proved to be almost equal to the outside pressure and it was, therefore, necessary to weight down the base slab in order to be able to pump the cofferdam dry without danger of a " blow." The weighting-down was carried out in sections by constructing temporary bulkheads across the cofferdam and retaining water inside over such portion of the dam where work was not proceeding on the pier proper. In addition, kentledge was used to increase weight. After pumping dry, the main pier was constructed in the normal manner.

Embankments

The embankments ashore presented no particular problem, but along the line of the embankment over the tidal flats between the north abutment and the scour span the natural river-bed consisted of approximately 14 feet of silt with thin layers of peat. A narrow embankment of ballast filling, approximately 20 feet wide at the top was placed alongside the existing north approach and the desired effect of gravity displacement was obtained insofar as this narrow embankment sank at each low tide, sometimes as much as 5 feet, and pushed the soft material sideways. The embankment was continually topped up and filling was discontinued only when movement stopped owing to a stable condition of embankment and heaped-up silt at the toe having been reached. Probings showed that this sliding occurred in accordance with orthodox theories. When the movement stopped the embankment was widened by a further 10 feet approximately without any new displacement resulting.

In order to break down the shear resistance which had arrested the gravity displacement, small-scale toe-shooting was then employed. Owing to the proximity of the new embankment to the old bridge and embankment, restrictions were placed on the number and size of the explosive charges to be employed and the ideal explosive for the purpose was not readily available. As a consequence the two comparatively small-scale attempts which were carried out proved abortive. It was decided, therefore, to remove the remaining soft material (in all, approximately 10,000 cubic yards) by dredger down to the underlying gravel strata and to fill the resulting trench with imported ballast.

Bridge deck

Following the placing of the beams on beech blocks on the piers, the service pipes were laid on the diaphragms. The large gas pipes, which were supplied in long lengths giving two joints only for each span, were rolled in under the Tee-head from the top. In order to avoid interference with navigation, all staging for the construction of joints in the deck and for the transverse stressing and construction of the parapet was hung from the bridge deck through temporary holes left in the narrow joint between beams.

The knitting together of the bridge deck commenced with the construction of the solid diaphragm over the piers. The soffit of this diaphragm which was required to be clear of the pier top, was formed as a precast slab laid on $\frac{3}{4}$-inch-thick fibre boarding which, together with the beech blocks temporarily supporting the beams, was raked out after transverse stressing of the diaphragm.

The construction of the pier diaphragms was followed by the jointing up and stressing of the lower diaphragms, the placing of the junction slabs and jointing and stressing of the deck proper.

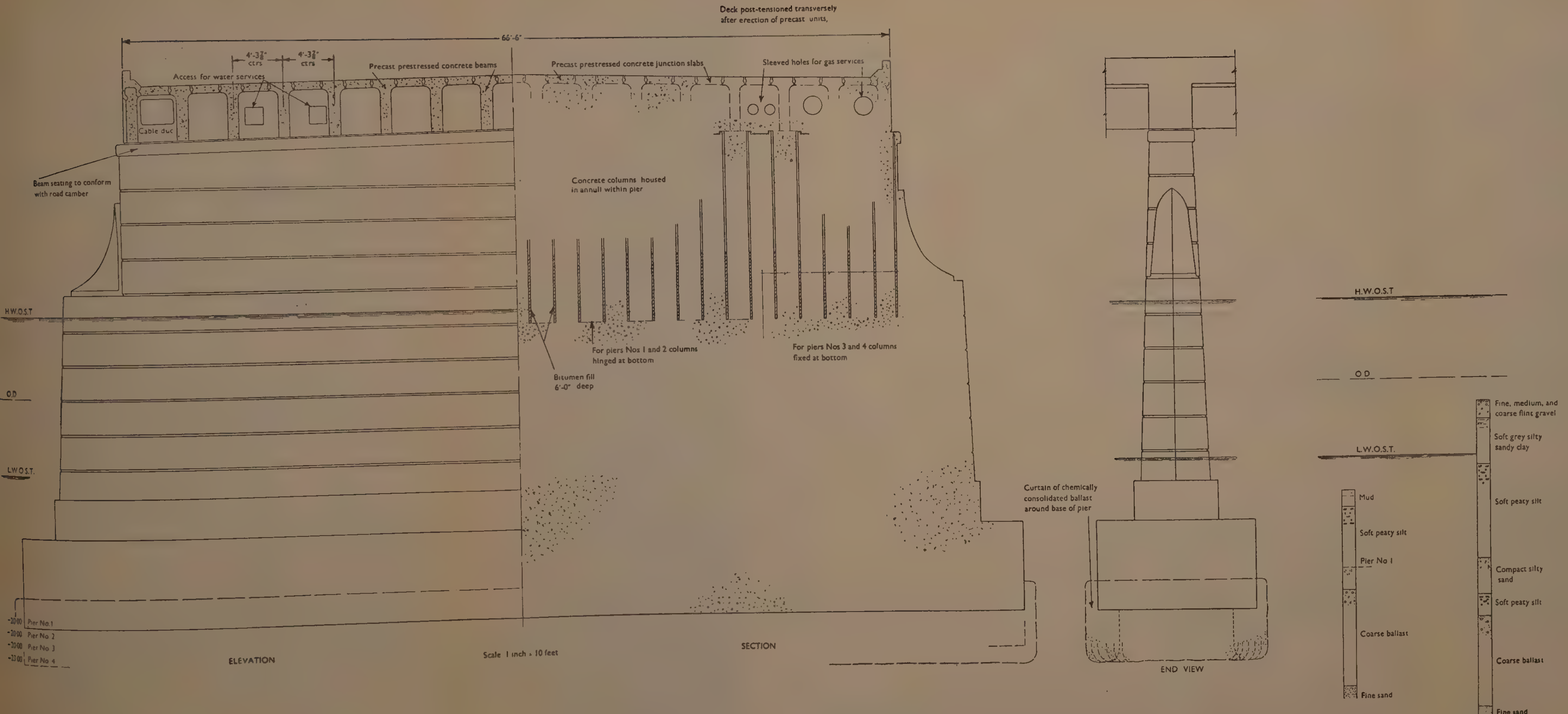
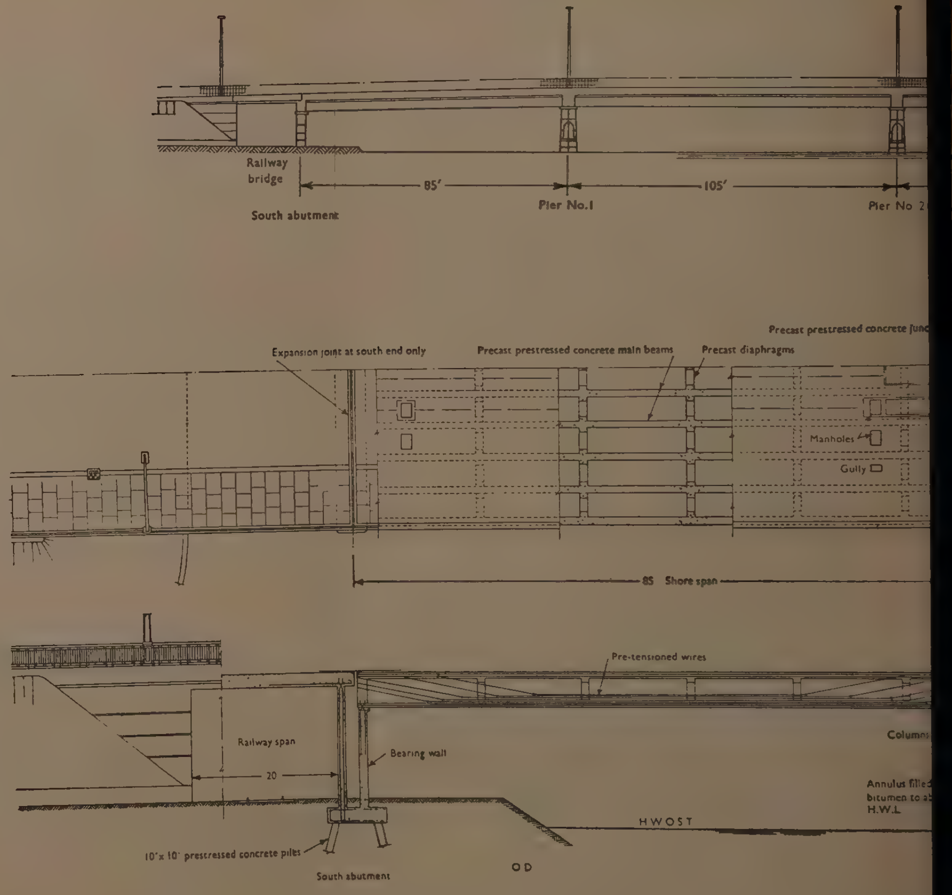


FIG. 4.—BRIDGE PIERS

NEW NORTHAM BRIDGE, SOUTHAMPTON



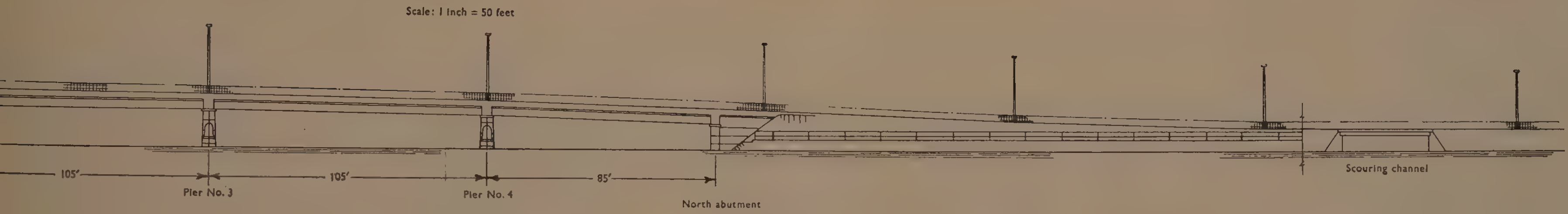


FIG. 6.—ELEVATION OF BRIDGE

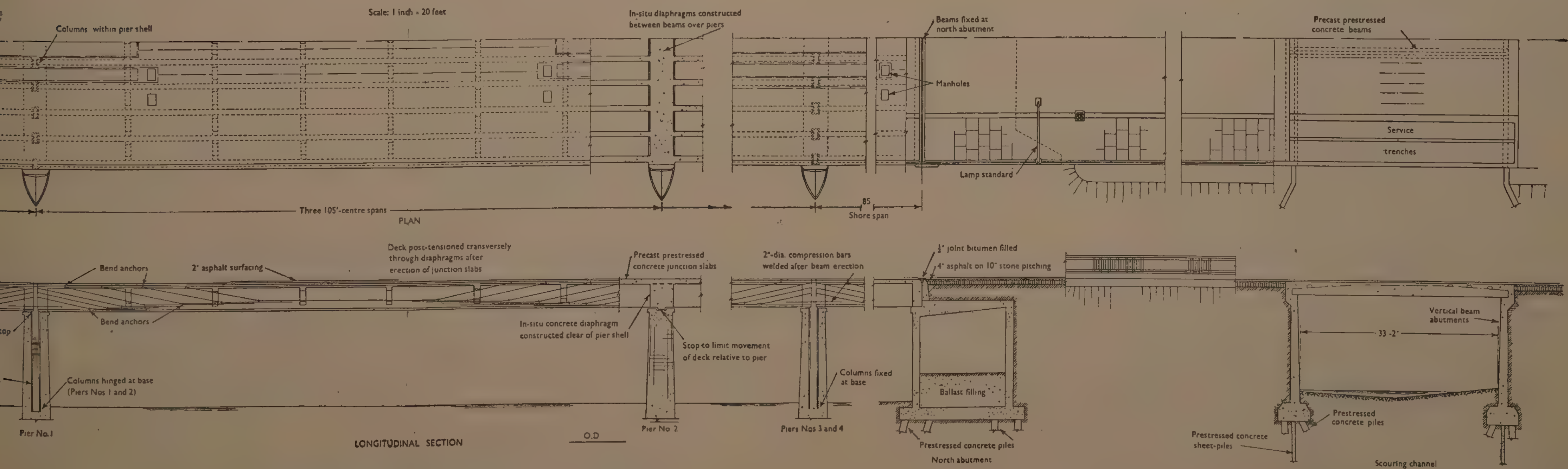


FIG. 7.—STRUCTURAL ARRANGEMENT OF BRIDGEWORK

F. L. WOOLDRIDGE, JOHN CUEREL,
and K. R. HAUCH

Prior to jointing a rubber tube was passed through all transverse holes in the beams and junction slabs and inflated. The tube was deflated and withdrawn immediately after the jointing concrete had set and the Freyssinet cables were threaded through. Shuttering to the soffit of the joints was hung from the top by a stainless wire which was withdrawn immediately after the concrete set.

Work proceeded from south to north finishing with the welding up of the anchor bars, fixing the now continuous deck to the north abutment.

General

Work commenced on the site on the 1st April, 1952, with the temporary structures for the pier construction and the construction of the stressing pit. The first beam was cast in December 1952 and the last in March 1954. In April 1954 traffic was diverted on to the northbound carriageway north of the scouring span in order that work might proceed on the southbound carriageway overlapping the existing road. On the 15th July, 1954, all traffic across the old bridge was brought on to the northbound carriageway of the new bridge and work commenced on the road overlapping the old road at the south end of the job. This work was completed by the end of September 1954.

Demolition of the old bridge commenced at the beginning of August 1954 and is scheduled for completion by December.

The work was carried out on behalf of the Ministry of Transport and the County Borough Council of Southampton.

Engineers associated with the execution of the work included the following assistants to the Authors :

Borough Council :—
Mr H. Simpson, B.Sc.(Eng.), A.M.I.C.E.

Consulting Engineers :—
Mr F. I. Childs, A.M.I.C.E., Senior Engineer, Mr C. W. Pike, A.M.I.C.E., Resident Engineer, and Mr L. G. Ellis, B.Sc.(Eng.), A.M.I.C.E.

Contractor :—
Mr P. J. Harder, M.Sc. (Copenhagen), sub-Agent.

The Paper, which was received on the 6th October, 1954, is accompanied by nine photographs and eleven sheets of drawings, from which the half-tone page plates, folding Plates 1 and 2, and the Figures in the text have been prepared.

Discussion

Mr H. C. Adams observed that the Northam Bridge had been constructed on the most up-to-date principles, in spite of the fact that its design had been started 5 or 6 years previously.

The timber bridge built about 1800 had a width between parapets of 24 ft which was still sufficient for the wrought-iron bridge built in 1889, the cost of which was given as £9,000. The width of the present bridge was 65 ft between parapets and the total cost was sixty or seventy times that of the previous bridge. Those differences drew attention to the size of the problem that faced Britain today. In 1889 the vast majority of bridges had already been built and were still being used. The problem of their reconstruction, so far as engineering was concerned, was not very great; all the material and technical resources were available. The problem was entirely a financial one.

There might be members present who thought that they had just as pressing claims as Southampton had; he called attention to a statement made in the Paper: "From the date it [the wrought-iron bridge] was taken over by the Council, it was realized that the structure did not come up to modern standards." That was surely a masterpiece of understatement!

Mr Adams referred to the chemical consolidation around the base of the foundation forming a box to reduce the spread of the foundation bed, and to the method of sealing the cofferdams and keeping track of the water pressure below the base. The flexible columns inside the piers reminded one of Waterloo Bridge, for which the same consulting engineers had been responsible.

Referring to the feature of continuity over the whole of the bridge Mr Cuerel had said, when introducing the Paper, that he was not sure if there was any appreciable economy in making the structure continuous. Mr Adams suggested that, even if there had been no special economy in that, so long as there had been no appreciable loss in using that feature there was a very big advantage in eliminating the expansion joints over the piers. It was in such joints, whether they were expansion joints or merely construction joints, that the flexing and the resultant opening and cracking and fretting away of the wearing surface allowed infiltration of water with the consequential troubles that involved. It was a problem that arose in many bridges, and it was a very difficult one to deal with. The advantage of eliminating that problem at the beginning was very important.

By arranging the flexible piers monolithic with the superstructure the consulting engineers had managed to do away with any form of rollers or small rockers, which often caused trouble. There were no rockers even at the end of the bridge, where there was a single expansion joint

FIG. 16.—AERIAL VIEW OF SITE

FIG. 17.—AERIAL VIEW SHOWING BRIDGE SITE AND CASTING YARD

Fig. 18.—18-inch gas main slung in position

Fig. 19.—Aerial view of partly completed bridge

It would be interesting to know more about the expansion device in the roadway surface—a matter which had been very well managed in the Northam Bridge.

The bend anchors were an interesting feature which had enabled heavily stressed wires to be concreted into the beam without the need for subsequent grouting. It was always a big advantage if that could be done. During the examination of the beam which had been tested to destruction, had there been any opportunity to break away the concrete and find out the condition of it at the points where the heavily stressed bars had their bends? Could the Authors give any information on that point?

Could the Authors say how an estimate had been made of the amount of transverse prestressing that was required in the deck?

It was interesting to note that ordinary Portland cement had been used in preference to rapid-hardening cement. It had been quite satisfactory and obviously cheaper, and had considerably reduced the shrinkage of the concrete—a problem which always had to be faced.

He thought the reference to loading on p. 278 was perhaps not precise; it stated: "The bridge was required to be designed to carry Ministry of Transport Standard Loading plus Abnormal Loading Class C." He thought that what was probably meant was that the bridge had been designed for Ministry of Transport normal loading and then, as an alternative, checked against abnormal loading; i.e., there was not the addition of the two, but both conditions had to be met in that case as alternatives.

Referring to the raking out of the beech blocks underneath the beams at the tops of the piers he wondered if it had been quite as simple an operation as the wording in the Paper suggested.

Mr C. W. Pike showed a number of lantern slides to amplify some of the points mentioned in the Paper.

Fig. 16 was a view of the area rather than of the site itself. It showed how close to the old bridge the new bridge had been sited. On the Southampton side, the bridge alignment joined on to the existing road as shown (A). On the north side it carried right through as indicated (B), the whole of the outer stretches being embankment, to a total length of about 2,000 feet. The scour span mentioned in the Paper was shown. Its purpose was to preserve the foreshore levels in the timber pond upstream of the bridge. The embankment over the tidal flats between the scour span and main bridge was also shown. The railway, which had been diverted, originally ran behind the cinema, straight across the road at D and on through the works area. The diverted route was from a point behind the cinema, over the made-up foreshore to the railway span E, and thence *via* a cutting through the old bridge approach.

Fig. 17 showed the casting yard (A) in which the beams had been made and the journey they had had to make out to the bridge. First, they had

had to cross the existing railway line which had been still in use, so that a temporary removable bridge had been made necessary. The beams had been taken into a stacking area and subsequently to the landing stage, where they had been picked up by the derrick barge and transported to the job.

Fig. 18 showed one of the 18-inch gas mains slung over the side of the beam. Each pipe had been slung before the next beam was brought in. The slings consisted of mild-steel straps anchored well back into the deck; one of them could be seen at A. The straps had been so arranged that they remained in position holding the pipe clear of the diaphragms until all the transverse tensioning was completed, leaving just a small amount of concrete to be placed after the removal of the strap.

Fig. 19 was an aerial view of the partly completed job. Mention was made in the Paper of the 24-foot width of the old bridge compared with the 65-foot width between parapets of the new bridge. Fig. 19 showed that difference clearly. As a partly-constructed job, there could be seen the gaps where the flanges of the beams were curtailed for the insertion of the continuity slabs over the pier-heads. The longer gaps (A) were to allow the insertion of manholes for access to the services which were laid beneath the deck.

In the Paper it was stated that the cost of the previous wrought-iron bridge was £9,000. Mr. Pike believed that that was the cost merely of the bridge without any embankments. The £600,000 cost of the new bridge was an inclusive cost covering the compulsory purchase of ground, bridge construction, and about 2,000 feet of embankment construction. Therefore, the true comparison was rather different from the impression given.

Mr Donovan H. Lee asked whether the approximate cost of the superstructure per square foot of deck had been calculated; was there any similar calculation available for the sub-structure; and the chemical consolidation having been done, did the Authors consider that it had been good value? Cases where chemical consolidation had been valuable were well known but, in looking at the drawings, he could not help wondering what had caused the piling scheme to be abandoned.

On the question of transverse prestress, bearing in mind the recent prominence given to the question of adequacy of transverse prestress (see "*Structural Engineer*," March 1954, for instance), did the Authors consider the cost of additional transverse prestress not justified? Mr Lee added that the transverse distribution in that case did not appear to be below the average.

He noted that the Authors had used a concrete mix of $1 : 1\frac{1}{2} : 3$. Both German and British practice mostly used less cement, resulting, it was fully believed, in less creep, but that would be offset by the unusually low water/cement ratio (0·3) used. He presumed that rounded gravel had been used. In his experience, notwithstanding what had been

published on concrete mix design, it was best for engineers to avoid angular coarse aggregates if they could. Had any plasticizer been used in the concrete? Plasticizers were rather popular in Germany and the United States.

Mr Lee agreed with the use of the ordinary Portland cement, but thought it had not been much used previously for pre-tensioned work. Possibly the high early strength that had been obtained was a justification. He agreed that shrinkage stresses were greatly reduced by using ordinary Portland cement.

Mr F. Irwin Childs described some interesting features of the bridge which were not apparent on a cursory examination of the completed structure.

The scantlings of the beams proclaimed that the members themselves were prestressed, and in order to achieve an economic section it had been necessary to accommodate a compact arrangement of wires, particularly taking into account the necessity to provide for reversal of stress at the ends. Fig. 10, Plate 2 showed the arrangement of the wires which, though familiar enough as a pattern for normally reinforced concrete, was unusual for prestressed work. The details of the bend anchors, as well as the arrangement of the wires, had been provided by the consulting engineers, and the hinge for locating the bend anchors automatically in their correct position when the wires were tensioned had been devised by the contractor. In fact, the way in which the contractor had entered into the spirit of the design had been one of the happy features of the contract. From the consultants' point of view, another commendable example of this co-operation was represented by the decision to adopt a water/cement ratio of 0·3 for the concrete in the beams, which had been well below the maximum permitted by the specification.

The achievement of continuity for structures composed of precast prestressed members generally presented some problems, particularly when the construction depth was limited, and there was often a great temptation not to attempt it and to leave spans simply supported. In the present instance the solution had been provided within the depth of the deck slab by using precast portions of the slab as continuity members and placing them between the main beams. He considered that Mr Cuerel's analogy of the steel cover-plate was a very interesting and suggestive one.

Referring to the structural arrangement, it would be recalled that the Paper described the method by which the bridge deck was anchored at the north abutment, supported on flexible columns within the four river piers, and finally carried at the south abutment on a flexible wall in such a manner that a limited freedom of movement for temperature effects could be accommodated. Mr Childs showed a number of slides illustrating the construction of the piers and abutments. It could be seen that between

the columns at the pier head there was a key-way which engaged with a key formed on the bottom side of the in-situ diaphragms over the piers to limit movement; each column had a copper flashing around to protect the annulus, which was only partially filled with bitumen. The function of the shrouding of the upper part of the piers was threefold: first, to protect the columns, secondly, for aesthetic reasons, and thirdly, to encourage the travelling public to use the bridge, which was at least one of the reasons for which it had been built!

Mr G. O. Kee said it appeared to him that the transference of load from the beams to the pier was carried out by transferring it from the beam to the in-situ diaphragm which ran over the columns. The ends of the beam were smooth and had only the short ends of wires and two 2-inch-diameter bars protruding. Was that considered to be sufficient to transfer the reaction from the beam on to the in-situ diaphragm? The top continuity slabs, of course, also bore on the diaphragm. Was it considered that any of the load was transferred through them?

He did not agree with the comment on p. 280, regarding a post-tensioned scheme, that "it was feared that the losses might be such as to defeat the design." The quantity of post-tensioned work being carried out at the moment surely was a refutation of that comment.

Mr A. Goldstein asked what would happen in the case of the 18-inch gas main if any replacement were required. His office had recently had occasion to discuss that matter with one of the provincial gas boards, and it appeared that those mains did sometimes leak. From the construction details given in the Paper it looked as if it might be difficult to get a faulty section of the main replaced without dismantling and removing a number of additional sections.

Mr J. R. Lowe said that one of the most useful things about such a Paper was the opportunity it gave for criticism and discussion at a later date of what might be improved in the design. Considering that the bridge had been designed 6 years ago, he considered that it was very good indeed, but he felt that nowadays consideration might be given to post-tensioning with precast units. He thought that using 0·276-in.-dia. wires and bringing all the wires into one cable would probably give much better placing of the concrete. He knew that that would mean a certain increase of dead-weight which, he believed, was some objection, but it would represent a gain in that there would not be so much worry about shrinkage losses. He thought that nowadays there would be little difficulty in grouting up afterwards.

Another question that he wished to ask was why such a low water/cement ratio had been used. He thought it would have been much better to have used a concrete with a higher water/cement ratio, which was allowed, provided that the strength could be obtained. It would have been much easier to place.

He thought he had understood Mr Cuerel, in his opening remarks, to say that the cover-plates ought now to be made standard. Mr Lowe considered that there were other methods that were very suitable. Bent cables or bars could be used and ought to be considered.

With regard to the abnormal load, he asked whether or not any tension had been allowed in the concrete. He believed it had been stated that a figure of 15% had been used for losses. Had any attempt been made to check the shrinkage and creep losses afterwards ?

*** **Mr A. D. Holland** referred to the statement on p. 280 regarding losses, which had been assumed for design purposes to amount to 15 tons per square inch. That represented 21·4 per cent of the initial stress of 70 tons per square inch and the Authors had said that observations which they had taken indicated that that allowance was generous. Information of that nature was of value to designers when considering the extent to which provision should be made for loss of prestress, and it would be useful if the Authors could indicate the basis for the conclusion which they had formed.

The extent to which the variation in concrete strength could be minimized in the production of high-quality concrete at bridge sites was another matter where it was helpful to know of results which had been obtained in practice. On p. 281 it was stated that concrete strengths of 5,000 lb. per square inch and 7,000 lb. per square inch had been obtained at the date of release and at 28 days respectively. Presumably, in both cases those were minimum strengths and it would be interesting if the Authors would say how they compared with the mean strengths which had been obtained.

Mr Wooldridge, in reply, agreed with Mr Adams's remarks about the co-ordination that there had been on the job. There had indeed been complete co-ordination. He had not said very much about the condition of the old bridge, but there had been a time-limit for getting the new bridge built. There had been co-ordination on the part of everybody who had had anything to do with the bridge—the harbour board and in particular the engineers to public utility authorities. They had had to learn a new technique, and the fact that they had done so was now a matter of history.

Mr Pike had referred to the cost of £9,000 for the old bridge. Obviously the original embankments for the timber bridge had still been there, so that that figure had not included the embankments. The sum of £9,000 had been for the wrought-iron structure only. As Mr Cuerel had said, the figure of £600,000 for the new bridge included compensation for land, demolition of the old bridge, construction of the railway track, a new car park for the cinema, and so on. What the actual cost of the new bridge was he was not going to say at the moment. They had not paid the final

*** This contribution was submitteed in writing upon the closure of the oral discussion.—SEC.

account. And for that reason he was not going to give any breakdown figures.

Mr J. Cuerel, in reply, thanked Mr Adams for his remarks and for pointing out an error in the paper; the bridge had been designed for the standard loading with a check for the abnormal load.

Not as many critical guns as he had expected had been fired off. With regard to the concrete, there had been some criticism of the low water/cement ratio; that was a novel experience. The development of the mix had been fully described in the Paper, and he could only refer those who had raised this point to the Paper. Obviously, they had wanted the lowest possible water/cement ratio consistent with compaction in the work; the ceiling had been set at 0·4, and it had been found possible to work down to 0·3, helped by the rounded aggregate. Notwithstanding the very low water content, the maximum size of the aggregate and the large number of closely spaced wires, full compaction had been realized and a better example of prestressed concrete did not exist. Plasticizers had been tried at the beginning and had been discarded as being ineffective. Ordinary Portland cement had been found to give an early enough time for release, and its use had led to fewer shrinkage troubles.

The question of cost had been largely dealt with; the total of £600,000 included acquisitition of land and property, long approaches, road works off the site, and so on; and the actual cost of the bridge proper was much less than half of that total sum.

Reference had been made to the chemical consolidation and to the abandonment of the piling scheme for the piers; these matters had been covered in the Paper, which was one of such wide scope that it just was not possible to go into all the details fully.

Questions had been asked with regard to the transverse prestressing; the amount had been computed in a simple and rather generous manner on the basis of the dispersion required. He was aware that theories were being propounded as to what the transverse prestress should be; but theories were no better than the assumptions, explicit and implicit, on which they were based. Some time ago a bridge deck had been tested and it had been clear to him that the results confounded the theories—notwithstanding that the bridge tested did faintly conform to a fraction of the assumptions.

There had been protests from the post-tensioning interests about his remarks in the Paper on the considerations leading to the choice of the stressing system. In anticipation, he had exercised great care in the writing which, unfortunately, had not been matched in the reading. His remarks had been made for the benefit of engineers who were prepared to consider the relative merits of the different systems and to judge them in an unbiased manner. The experience had strengthened his opinion.

Various questions had arisen as a result of mis-reading the Paper and

misunderstanding of his opening remarks. Dealing with these and sundry others, he observed that the transference of load from beams to pier was described in the Paper, the end portions of the beams were *embraced* by the diaphragm and keyed thereto; the 2-inch bars had no function in that respect. He had not said that cover-plates should be standardized but that, depending on circumstances, similar devices had become accepted practice; the use of cables or bars had been considered as he had explained. When checked for abnormal loading a small tension was calculable, but he doubted if any would in fact arise. As stated in the Paper, the loss of stress in the steel had been assumed to be 15 tons per square inch (not 15 per cent), observations indicated that 10 tons per square inch would have been ample in the circumstances obtaining. The gas mains could be replaced in a simple manner using a floating stage.

Finally, he would like to place on record that in all his experience of civil engineering he had not been associated, in any capacity, with a happier job than the construction of New Northam Bridge.

Mr Hauch, in reply, referred to Mr Adams's remark that a great deal more could be said about the cofferdams and the underwater concrete in the piers. The system of underwater concreting was one which Mr Hauch's Company favoured and which he thought was extremely useful, particularly where sub-soil conditions such as those at Northam existed. It should always be remembered that water exerted its full pressure and no chances could be taken. They had kept a check on the pressure underneath the concrete base by means of the pipes described in the Paper and that simple gadget had proved its worth in indicating when there was a danger of the bottom lifting. Had the foundation lifted and the bottom been disturbed they would have run into considerably more trouble.

With regard to Mr Adams's remark on the bend anchors in the beam which had been tested to destruction, none of the bend anchors had, in fact, been cut out. They had cut back from the point of fracture to both sides for some distance until the concrete had been found to be intact. They had not reached the first bend anchor and there had been no marks on the beam to show that at the position of the bend anchor anything particular had happened.

Mr Adams had asked how the beech blocks had been removed. Before a decision on the use of those blocks had been taken a couple of the blocks had been tested by compression in a testing machine and it had been found that if they were made $\frac{5}{8}$ inch thick they would compress under the load to approximately the $\frac{1}{2}$ inch required. It should be remembered that although the beech block had been compressed it was not at the time of removal under compression, for the beam was then carried through the columns on to the pier. Although the wood had been compressed it would not close up on the tools used for removing it. They had, therefore, used an electric drill of a fairly small diameter and drilled several holes through

the blocks, after which it had been possible to rake out the bits between the holes.

The question of the water/cement ratio had been dealt with by Mr Cuerel and Mr Irwin Childs. The question was in part related to the matter of Portland cement versus rapid-hardening cement. It was a complex question but the major consideration was reduction of shrinkage. There had been found, in the first beams cast with rapid-hardening cement, a very considerable number of cracks. After using Portland cement cracks still occurred but to a smaller extent. Had they used a higher water/cement ratio or rapid-hardening cement more trouble would have ensued so they had used the lowest possible ratio that could be worked into the beam. It had been extremely hard going but by severe vibration of the concrete and by exact control the beams had been cast. On several occasions two beams had been cast in one day but for most of them it had taken 6 or 7 hours per beam in order to make absolutely sure that the concrete was vibrated right down to the bottom.

Additions to the concrete had been tested both with regard to strength and workability and it had been found that there was very little difference when using the low water/cement ratio that had been decided upon so no additives had been used, first because they did not appear to be economical, and secondly because he was convinced that additions to concrete were not sound, whether they were a help in placing or not.

CORRESPONDENCE on the foregoing Paper is now closed and no contribution, other than those already received at the Institution, will now be accepted.—SEC.

Paper No. 6034

A RATIONAL APPROACH TO THE DESIGN OF DEEP PLATE GIRDERS

by

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and

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(*Ordered by the Council to be published with written discussion*)

SYNOPSIS

The Paper reviews the present position of specifications relating to the design of plate girders, and deals with the buckling of plates in shear and bending. Rules are suggested for the design of web plates considering (*a*) elastic buckling, or (*b*) failure by yielding. These rules relate the permissible stresses for combined shear and bending or combined shear and direct stress. A review of experimental work is given, including the effects of continued loading after elastic buckling has occurred. Permissible stresses, with appropriate safety factors, are then discussed in more detail, and design curves showing these for various load and edge-support conditions are presented. Theoretical and empirical rules for horizontal and vertical stiffeners are compared, and recommendations for the design of these for webs governed by either elastic or yield conditions are put forward. An example from practice is worked to show the application of the suggested design procedure, and a bibliography is included.

INTRODUCTION

THE problem of the design of web plates and stiffeners for deep plate girders is one which has received comparatively little attention in Great Britain, although in other countries there is a considerable amount of literature on the theoretical and experimental aspects of the subject. Recently, Rockey [1] has presented a valuable contribution to British literature on plate-girder design, showing how the theory of elastic stability may be used in the design of plate-girder webs and stiffeners, and reviewing experiments on plate girders from 1845 to the present day. Another notable Paper was that presented by Moissieff and Lienhard,[2] which contained numerous design rules, some of which are semi-empirical.

Neither of the above Papers was confined in scope to steel. The present Paper deals specifically with steel girders, for it is assumed that the

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[1] The references are given on p. 333.

material has a clearly defined yield point, up to which Hooke's law is applicable. Many of the results presented may be applied to girders of aluminium alloy, provided that the differences in the stress/strain curves are taken into account.

This Paper has been written as a result of problems which presented themselves in the design of the main plate girders of two large bridges over the Tigris at Baghdad. They were designed on the cantilever-and-suspended-span principle, with depths ranging up to a maximum of 18 ft 6 in. at the piers, where the girders were subjected simultaneously to maximum shear and bending moment. The Authors were faced with the choice of designing the girders in accordance with the regulations then in force (in 1952) or of finding some alternative basis of design in the interests of economy.

The fundamental principles essential for this purpose are those of the theory of elastic stability. The importance of this theory has now become more widely recognized, and formulae based thereon are being incorporated in design specifications, so it is important that its applications should be widely known and understood.

Review of existing design rules

Prior to the publication in 1949 of the Code of Practice for Simply Supported Steel Bridges, the design of steel bridges in Britain was governed by B.S. 153 : 1937. Both of these laid down the rule that the thickness of the web plate should not be less than 1/180 of the clear distance between the flange angles, i.e., the depth/thickness ratio of the web is limited to 180. This rule has the effect of producing thick webs in deep girders. It should be noted that B.S. 153 limited the shear stress in webs to 5·5 tons/sq. in. (irrespective of the d/t ratio)* whereas the Code introduced a rule (Clause 220 (a)) varying the permissible stress according to the dimensions of panel and the d/t ratio; e.g., the stress allowed at $d/t = 120$ is 5·5 tons/sq. in., whilst for $d/t = 180$ it becomes 1·9 ton/sq. in., where d denotes the minimum clear distance between flange angles (or flange plates where there are no flange angles) or vertical stiffeners or, in the case of horizontal stiffeners, the minimum clear distance between flange angles and the horizontal stiffeners, whichever is the lesser, and t denotes the thickness of the web.

It is interesting to note that this Clause recognizes, for the first time in British practice, the use of horizontal stiffeners, although these have been used in large bridges on the Continent for 20 years. However, as will be shown, the provisions of the Clause are inadequate for webs of girders with horizontal stiffeners, and no rules are formulated therein for the design of the latter.

For a girder subject to both bending moment and shear, B.S. 153 : 1937

* The Authors understand that B.S. 153 is at present being revised.

does not reduce the shear stress to allow for the effect of bending. Clause 220 (b) of the Code of Practice requires the allowable shear stress to be reduced by a percentage $p = kf_b^2$, where f_b denotes bending stress in the flange in tons/sq. in., and k varies from zero at $d/t = 70$ to 1·23 for $d/t = 180$. Thus, for a flange stress of 8 tons/sq. in., the reduction, for $d/t = 180$, would be 79%.

Comparing the Code with B.S. 449 (Amendment 1), the latter limits the d/t ratio to 170 for steel to B.S. 15, 160 for steel to B.S. 968, and 150 for steel to B.S. 548. The permissible stress is given in terms of the side ratio and slenderness ratio, thus :

$$F_s' = \left(\frac{225}{b/t}\right)^2 \left[1 + \tfrac{3}{4}\left(\frac{b}{a}\right)^2\right] \text{tons/sq. in.} \quad . \quad . \quad . \quad . \quad (1)$$

where a denotes the greater unsupported dimension of the panel, b the lesser unsupported dimension of the panel, and t the web thickness. An upper limit for F_s' is imposed (6·5 tons/sq. in. for steel to B.S. 15). When shear is combined with bending, the shear stress is reduced as in the Code.

In contrast to the specifications referred to, the current German regulations for both bridges and buildings do not limit the slenderness ratio of web plates. These regulations are stated in specification DIN 4114 adopted in 1952,[3] which covers the design of structural members, e.g., struts and web plates, in which considerations of elastic stability arise. DIN 4114 requires the calculation of a safety factor against buckling for the various panels into which the web may be divided, taking into account both shear and bending stresses.

Buckling of plates

When a uniform plate is subject to forces in its own plane, the plate may buckle out of its original plane. The onset of buckling depends upon the nature of the stress, the proportions of the panel, the conditions of edge support, and the elasticity of the material. Assuming Hooke's law to apply, the stress condition at which buckling occurs in a rectangular plate of side lengths a and b may be expressed in terms of the Eulerian stress σ_e for buckling of the plate as a strut simply supported at the sides of length a. Thus :

$$\sigma_e = \frac{\pi^2 D}{b^2 t} = \frac{\pi^2 E}{12(1 - \nu^2)} \cdot \left(\frac{t}{b}\right)^2 \quad . \quad . \quad . \quad . \quad (2)$$

where D $(= Et^3/12(1 - \nu^2))$ denotes flexural rigidity of the plate per unit width, E Young's modulus, and ν Poisson's ratio ($\simeq 0{\cdot}3$ for steel). The factor $(1 - \nu^2)$ occurs as a result of the absence of lateral strain in a wide plate.

(a) Buckling due to shearing stress

The "critical" stress τ_{cr} at which buckling would occur in a perfectly flat plate subject to uniform shear is given by:

$$\tau_{cr} = k_s \times \sigma_e \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

where k_s denotes the buckling coefficient for shear, depending on the boundary conditions and the side ratio α $(= a/b)$.

The effect of buckling on the plate is to form a chequer pattern of rough squares, the sides of which are approximately equal to the shorter sides of the plate, with crests and troughs in alternate squares. The buckling coefficients for rectangular plates in shear under various conditions of support will now be considered.

(1) *Plate simply supported on four sides.*—This case was solved first by Timoshenko [4] and later by Bergmann and Reissner [5] and Seydel.[6] From the results of their investigations it can be said that the constant k_s in equation (3) can be expressed in terms of α thus:

$$k_s = 4{\cdot}00 + \frac{5{\cdot}34}{\alpha^2} \text{ where } \alpha < 1 \quad . \quad . \quad . \quad . \quad . \quad (4)$$

$$\text{or } k_s = 5{\cdot}34 + \frac{4{\cdot}00}{\alpha^2} \text{ where } \alpha > 1 \quad . \quad . \quad . \quad . \quad (5)$$

For square panels, $\alpha = 1$ and $k_s = 9{\cdot}34$. (These formulae are embodied in DIN 4114.)

(2) *Plate clamped on two edges and simply supported on the other two edges.*—This case has been solved by Iguchi [7] for rectangular plates clamped on either the long or short sides.

(3) *Plates clamped on four sides.*—The solution in this case has been given by Iguchi, and by Moheit.[8] These differ somewhat, Moheit's solution being confirmed by the work by Wästlund and Bergmann.[9]

The results of these various tests and analyses (as quoted by Wästlund and Bergmann) are given in Table 1, p. 306.

(b) Buckling due to bending

The critical edge stress (within the elastic limit) for a plate of length a, height b, and thickness t, subject to bending moment applied to a plate acting in the plane of the plate can be found from:

$$\sigma_{cr} = k_b \times \sigma_e \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (6)$$

(1) *Plates simply supported on four sides.*—Timoshenko has found the value of k_b as a function of α, the minimum value being 23·9. These results are plotted as a series of curves tangent to the minimum

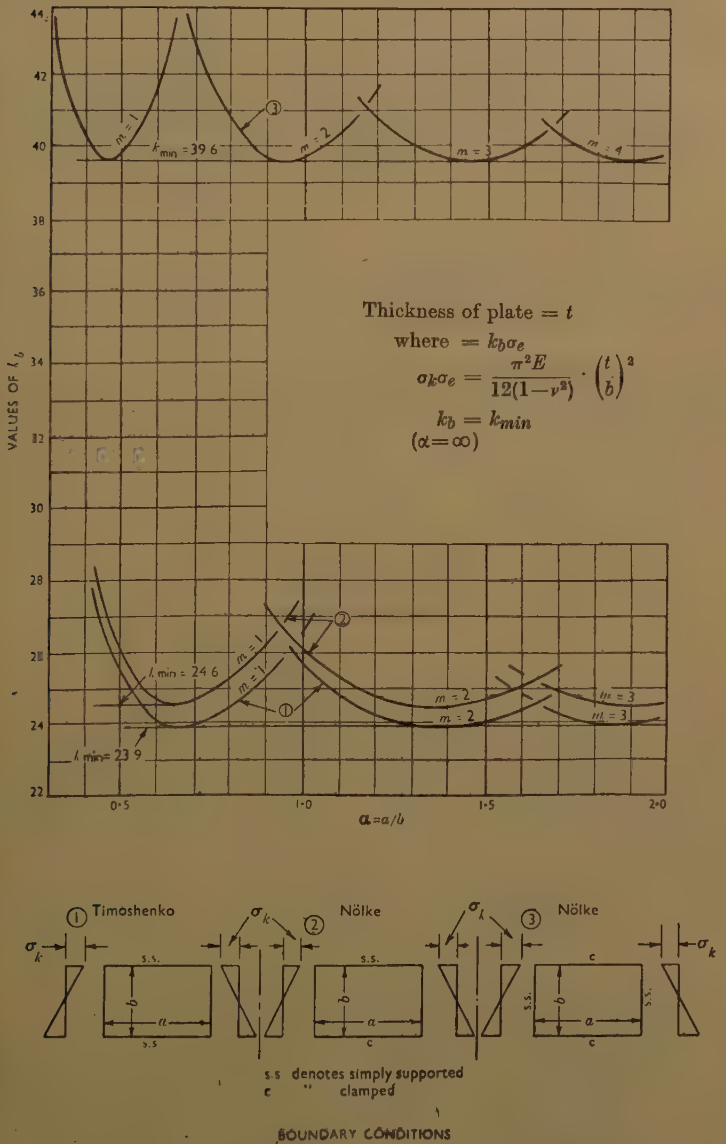


FIG. 1.—VALUES OF k_b PLOTTED AS A FUNCTION OF SIDE RATIO α FOR VARIOUS BOUNDARY CONDITIONS

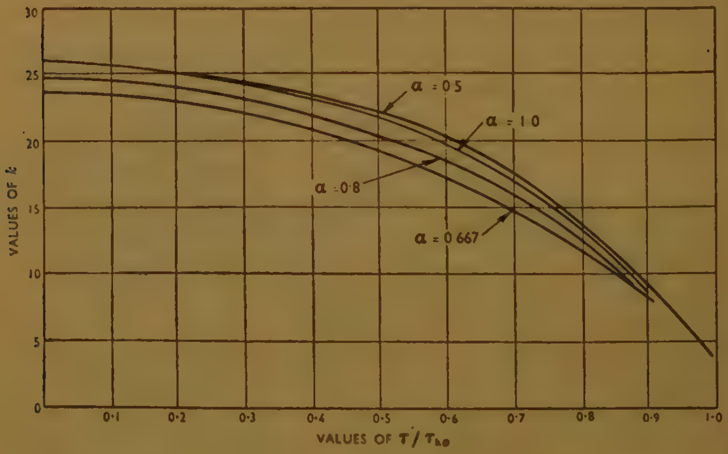
(Fig. 1). Each curve consists of several wave-shaped branches, th branches corresponding to the different numbers of waves of th deflexion surface.

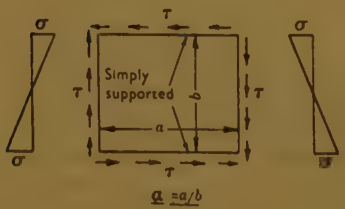
(2) *Plates clamped on side in tension, and simply supported on othe three sides.*—The solution of this case has been given by Nölke,[10] th minimum value of k_b being 24·48. The curves obtained by Nölk agree fairly closely with those of Timoshenko, and show that th effect of clamping the tension side is small (about 2·5%).

(3) *Plates clamped on sides in compression and tension, and simpl supported on other two sides.*—The solution in this case is also given b Nölke, the curve giving a minimum value of 39·6 for k_b. The effec of clamping the compression side is thus considerable.

Buckling due to combined bending and shearing stress

The problem of the buckling of a plate under the action of shearin forces and bending moments in the plane of the plate has been solved onl





Thickness of plate $= t$

τ_{ko} = critical value of shearing stress f the plate subjected to the action shearing forces only

For a given shearing stress τ the corre ponding critical bending stress is foun from $\sigma_k = k\sigma_e$

where $\sigma_e = \dfrac{\pi^2 E}{12(1-\nu^2)} \cdot \left(\dfrac{t}{b}\right)^2$

FIG. 2.—CURVES FOR DETERMINING CRITICAL STRESSES FOR PLATES SUBJECTED T COMBINED SHEARING AND BENDING STRESSES

in the case of plates simply supported on all four edges. Timoshenko and Stein [11] have given solutions based upon strain energy analysis. The results obtained by Timoshenko and shown in Fig. 2 are the more correct. Values of k_b are plotted for varying values of α as a function of τ/τ_{ko}, where τ denotes the actual shearing stress and τ_{ko} the critical shearing stress for a plate subject to pure shear.

Chwalla [12] has stated that the following relation is approximately correct in this case, irrespective of the side ratio α :

$$\left(\frac{\sigma_k}{\sigma_{ko}}\right)^2 + \left(\frac{\tau_k}{\tau_{ko}}\right)^2 = 1 \quad . \quad . \quad . \quad . \quad . \quad (7)$$

where σ_k and τ_k denote the simultaneous stresses producing buckling under combined loading, and σ_{ko} and τ_{ko} denote the respective critical stresses for bending and shear acting independently.

It is of interest that the above relation is similar to that obtained by the maximum-shear-stress theory for a material under combined direct stress σ and shear stress τ. This theory assumes that failure occurs when the shear stress on any plane reaches a value τ_s equal to half the direct yield stress σ_s obtained in a direct tensile test. Thus failure occurs when :

$$\sqrt{\left(\frac{\sigma}{2}\right)^2 + \tau^2} = \tau_s.$$

and since $\tau_s = \dfrac{\sigma_s}{2}$:

$$\left(\frac{\sigma}{\sigma_s}\right)^2 + \left(\frac{\tau}{\tau_s}\right)^2 = 1 \quad . \quad . \quad . \quad . \quad . \quad (8)$$

However, it is known from experience that the yield stress in shear is greater than half that in tension, and it has been suggested that equation (8) for the failing condition is valid if σ_s and τ_s represent the yield stress of the material, as determined by experiment, in tension and shear respectively.

A more exact expression relating the yielding of an isotropic material to the direct and shear stresses is known as the Hüber von Mises Hencky criterion for yielding. This is based on the maximum shear-strain-energy theory, and can be written for a two-dimensional system thus :

$$\sigma_s = \sqrt{\sigma_x^2 - \sigma_x\sigma_y + \sigma_y^2 + 3\tau_{xy}^2} \quad . \quad . \quad . \quad . \quad . \quad (9)$$

where σ_s denotes the yield-point stress for the one-dimensional state of stress, and σ_x, σ_y, and τ_{xy} the actual stresses at yielding in the two-dimensional system. This formula applies to a material having a well defined yield point, and has been found to agree closely with the results of a large number of tests, particularly those on steel specimens. Thus, for a material under pure shear, failure would occur when :

$$\sigma_s = \sqrt{3\tau_{xy}^2} \text{ or } \tau_{xy} = \frac{\sigma_s}{\sqrt{3}} \quad . \quad . \quad . \quad . \quad . \quad (10)$$

TABLE 1.—BUCKLING COEFFICIENTS FOR SHEAR

Boundary conditions	Authority	Values of k_s									
		$\alpha = 1{\cdot}0$	$\alpha = 1{\cdot}2$	$\alpha = 1{\cdot}4$	$\alpha = 1{\cdot}5$	$\alpha = 1{\cdot}6$	$\alpha = 1{\cdot}8$	$\alpha = 2{\cdot}0$	$\alpha = 2{\cdot}5$	$\alpha = 3{\cdot}0$	$\alpha = \infty$
All edges simply supported	Timoshenko	9·4	8·0	7·3	7·1	7·0	6·8	6·6	6·3	6·1	5·35
— do —	Eqns (4) and (5)	9·34	8·12	7·38	7·12	6·90	6·57	6·34	5·98	5·79	5·3
Clamped along short edges, simply supported on long edges . . .	Iguchi	12·28	—	—	7·78	—	—	6·70	6·40	6·17	5·35
Clamped along long edges, simply supported on short edges . . .	,,	12·28	—	—	11·12	—	—	10·21	9·81	9·61	8·99
Clamped on all edges .	,,	14·58	—	—	11·40	—	—	10·96	—	10·85	8·99
— do —	Moheit	14·74	—	—	—	—	—	10·42	—	—	8·96

This relation is approximately satisfied in the proportion of the usual allowable stresses in shear to those in tension or compression.

Rewriting equation (9) leads to the criterion for yielding:

$$\sigma_s^2 = (\sigma_x - \sigma_y)^2 + \sigma_x\sigma_y + 3(\tau_{xy})^2$$

hence

$$\left(\frac{\sigma_x - \sigma_y}{\sigma_s}\right)^2 + \frac{\sigma_x\sigma_y}{\sigma_s^2} + \frac{3\tau_{xy}^2}{\sigma_s^2} = 1$$

and substituting $\tau_s = \dfrac{\sigma_s}{\sqrt{3}}$

$$\left(\frac{\sigma_x - \sigma_y}{\sigma_s}\right)^2 + \frac{\sigma_x\sigma_y}{\sigma_s^2} + \left(\frac{\tau_{xy}}{\tau_s}\right)^2 = 1$$

When $\sigma_y = 0$, i.e., when the material is under direct stress in one direction only combined with shear stress, as in the case of a web of a girder subject to combined bending and shear, then:

$$\left(\frac{\sigma_x}{\sigma_s}\right)^2 + \left(\frac{\tau_{xy}}{\tau_s}\right)^2 = 1 \quad . \quad . \quad . \quad . \quad . \quad . \quad (11)$$

Equation (11) is identical in form with equation (8) based upon the maximum-shear-stress theory, but replaces $\dfrac{\sigma_s}{2}$ by $\dfrac{\sigma_s}{\sqrt{3}}$ as the shear yield stress. Comparing the expression (11) just obtained with equation (7) due to Chwalla, it may be seen that similar expressions can be used when examining a plate subject to combined bending stress and shear to determine (*a*) if, on the assumption that the plate remains elastic, its buckling stress has been exceeded, and (*b*) if, on the assumption that the material has a definite yield point, yielding due to combined stresses has occurred.

However, in dealing with deep girders where horizontal stiffeners are used, it should be borne in mind that three cases of direct stress may arise:

(1) pure bending, i.e., neutral axis lies within the panel;
(2) linearly varying compressive or tensile stress; and
(3) a condition approximating to pure compression or tension for a narrow panel adjacent to a flange.

In the present Code of Practice no distinction is made between the panels of a horizontally stiffened girder of these different types. If the overall-depth/thickness ratio is limited to 180, the criterion for failure of such panels due to bending would be yielding, not elastic buckling. If, however, the Code be extended to cover the case of overall-depth/thickness ratio $>$ 180, a distinction should be made between the different types of panel.

In order to illustrate this point, consider web panels of dimensions b and d, as shown in Fig. 3, each of which is subject to direct stress at the sides of length d. The three diagrams represent a state of stress of:

(*a*) pure bending (with the neutral axis at mid-height),
(*b*) linearly varying direct compressive stress, and
(*c*) pure compression.

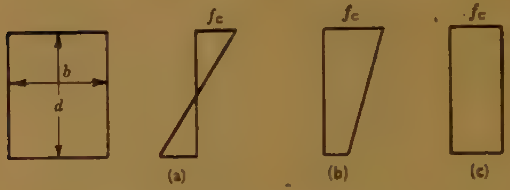


FIG. 3.—TYPES OF DIRECT STRESS IN WEB PANELS

Assuming simply supported edges, the coefficients determining the instability stress under the varying conditions in the formula: $fc_{cr} = k\pi^2 D/d^2t$ are given in Table 2.

TABLE 2

Side ratio b/d	0·4	0·6	0·8	1·0	1·5
k (pure bending)	29·1	24·1	24·4	25·6	24·1
k (varying compression min max $= \frac{1}{2}$)	10·8	7·1	6·1	6·00	6·1
k (pure compression) . .	8·41	5·14	4·20	4·00	4·3 app.

It is clear from the figures in Table 2 that the critical stress for pure compression is much less than that for pure bending. Hence, for exterior panels near the compression flange, the critical compressive stress will be less than for interior panels of the same dimensions subject to pure bending. When the direct stresses are combined with shear stress the same argument will still apply. Thus, the design method referred to, i.e., Tables II and III, p. 21, of the Code would appear to be optimistic if applied to a panel near the compression flange. This case has been dealt with by Timoshenko (see Table 36 on p. 355 of reference 4), and the variation of the buckling coefficient with the proportions of the panel is shown in Fig. 4. The buckling stress of a panel under linearly varying direct stress combined with shear may be studied with the aid of Fig. 5. The curve is approximately parabolic, and the relation may be written thus:

$$\frac{\sigma}{\sigma_{crit}} + \left(\frac{\tau}{\tau_{crit}}\right)^2 \not> 1 \quad . \quad . \quad . \quad . \quad . \quad (12)$$

With regard to the effect of combined stresses on yielding, the implication of the Code appears to be that, for shallow girders, the combination of stresses due to shear and bending does not lead to yielding, unless one or other of the yield stresses of the material for the individual loading conditions is exceeded, or that a reduced safety factor under the action of combined stresses is acceptable.

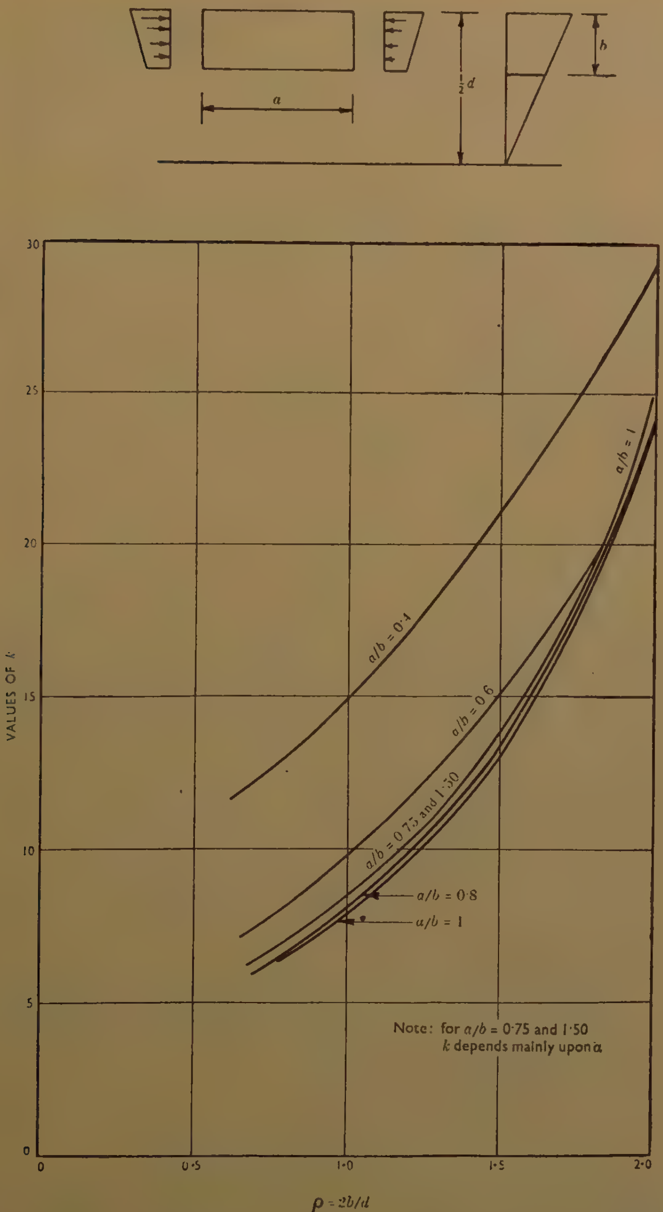


FIG. 4.—INSTABILITY COEFFICENTS FOR SIMPLY SUPPORTED PANELS SUBJECTED TO BENDING STRESSES

(Derived from Table 36, p. 355, reference 4)

$$\text{Inability stress } \sigma_{cr} = k\,\frac{\pi^2 D}{b^2 t}$$

$$D = \frac{Et^3}{12(1-\nu^2)}$$

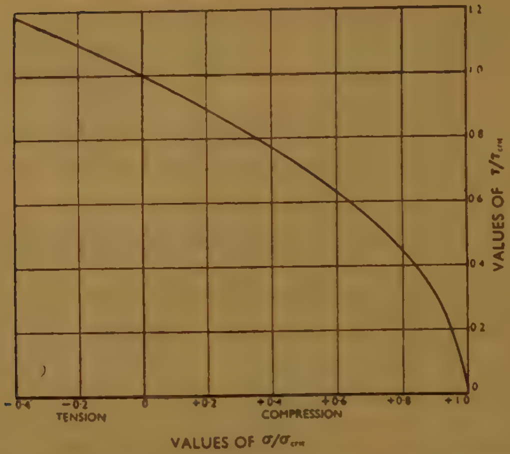


Fig. 5.—Buckling stresses for panels subjected to combined shear and linearly varying direct stress

Suggested rules for the design of web plates

If vertical stiffeners only are used, the danger of a web buckling in bending limits the depth/thickness ratio (when the stiffener pitch is more than about 50% of the web depth) to values such as those given in the Code of Practice. If, however, horizontal as well as vertical stiffeners are used, then, by suitable spacing of the former, it will be found that the buckling stresses become greater than the yield stress, i.e., it is possible to obtain maximum economy in material.

The Authors suggest that the aspects of both instability and yielding can be covered if the following method be used :—

(1) determine the instability stresses $F_{b\ \text{crit}}$ (bending) and $F_{s\ \text{crit}}$ (shear) for the panel considered, assuming E and v to be constant;

(2) divide these by a suitable factor of safety to find the permissible stresses F_b and F_s (with upper limits equal to the stresses applicable to a shallow girder).

Case (*a*).—Where the panel contains the neutral axis, or if there are no horizontal stiffeners :

$$\left(\frac{f_b}{F_b}\right)^2 + \left(\frac{f_s}{F_s}\right)^2 \not> 1 \quad . \quad . \quad . \quad . \quad (13)$$

where f_b, f_s denote the actual stresses in bending and shear respectively. This rule corresponds to Chwalla's (equation (7)) for buckling in the elastic range, and also equation (11) which relates the stresses for failure by yielding.

Case (*b*).—If the panel does not contain the neutral axis, the above rule is applicable only when both F_b and F_s are the upper limits *based upon yielding*. If, however, either F_b or F_s is determined by buckling, then the design rule is based upon relation (12), and becomes :

$$\frac{f_b}{F_b} + \left(\frac{f_s}{F_s}\right)^2 \ngtr 1 \quad . \quad . \quad . \quad . \quad . \quad . \quad (14)$$

In formula (14) F_b is essentially a compressive stress ; if the panel lies in the tensile zone, f_b may be taken as negative, thus raising the permissible shear stress. The advantage to be gained thereby is limited by the possibility of yielding, and hence the " sum of the squares " rule should also be applied, using the limiting stresses instead of F_b and F_s.

The evaluation of F_b and F_s is of primary importance ; this will be discussed in the light of experimental evidence in subsequent sections of the Paper.

Review of experimental investigations

Experiments carried out on plates having calculated instability stresses well within the elastic range, where special precautions have been taken to ensure that the plates are initially flat, have shown that the deflexions of such plates, perpendicular to their planes, remain small at first, but later increase rapidly in a manner suggesting the development of elastic instability.

Comparatively few tests have been made on panels where the condition of " simply supported " edges has been reproduced. One important exception is the series of tests carried out by Seydel on duralumin plates subject to pure shear. The curves of lateral deflexion plotted by Seydel confirm Timoshenko's theory of buckling of plates with simply supported edges under the action of pure shear. An example of Seydel's curves for deflexion is shown in Fig. 6. Similar results have been obtained by experiments on plates in pure compression.

At this stage, an important difference between the buckling of a plate from that of a compressed bar should be emphasized. In the case of a bar, large deflexions occur as the critical buckling load is approached, and these continue to increase until the bar fails by the development of excessive bending stresses or prohibitively large deflexions. On the other hand, for a plate in which buckling occurs large deflexions are evident as the critical buckling load is approached, but their rate of increase diminishes in the neighbourhood of the buckling load. Loads *in excess of the buckling load* may be safely applied to the plate until yielding of the web occurs. The curve indicating the relation between load and lateral deflexion is shown in Fig. 7. This important difference may be explained

by assuming the plate to be bounded by stiff edge members, which, whils affording the desired condition of support, e.g. allowing the edges to rotat for simple support, will resist direct forces acting at the edges of the plate Since the edges of the plate are fixed in position, the lateral deflexion of th plate involves an extension of its "middle surface," with correspondin tensile stresses (known as "membrane stresses"). The membran stresses are additional to the bending stresses corresponding to the curva ture of the plate. As the plate first deflects, i.e., as, the buckling load i just exceeded, the stresses in the plate, apart from the externally applie

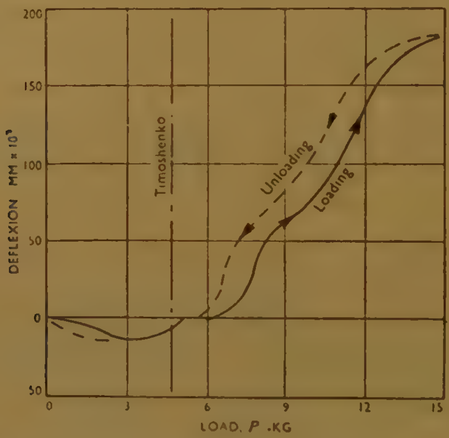


FIG. 6.—RELATIVE DEFLEXION AT CENTRE OF RECTANGULAR PLATE SUBJECTED TO SHEAR LOAD PLOTTED AGAINST LOAD P

(Plate size: $30 \times 15 \times 0{\cdot}02$ cm. Side ratio, $\alpha = 2$. P is the resultant of externa forces in direction of diagonal. The theoretical critical load according to Timoshenko (simply supported) is marked on the graph)

stresses, are mainly bending stresses. When the lateral deflexion becomes of the order of the plate thickness, the membrane stresses become appreciable. Large lateral deflexions cannot occur without high membrane stresses being induced, and the panel is thus restrained internally against the development of such deflexions. In virtue of this, the load applied to the panel can be increased after buckling has occurred; this may lead to an increase in deflexion, but will not result in failure until the combination of the various stresses acting on the plate causes yielding of the material.

EFFECT OF CLAMPING OF THE EDGES

Theoretically the buckling of plates with clamped edges will occur at stresses considerably higher than the buckling stresses for simply supported edges. It is, therefore, important to ascertain to what extent web plates

of actual girders may be regarded as clamped at their edges, either by flange plates or by stiffeners. Numerous tests have been carried out on typical girders, or panels representing these, of both riveted and welded construction. The results of these tests show that webs with riveted or bolted flange angles do not buckle at loads less than the corresponding critical loads for clamped edges, and in some cases the buckling load has been considerably higher.

Welded girders under test have behaved in some cases as if their edges were clamped but, in other cases, the webs have buckled at loads little higher than the critical loads for simply supported edges. This may be due to residual stresses resulting from the welding process. As the load is applied, local yielding at the weld may reduce the clamping effect of the flange plate.

The above results have been obtained from tests on webs subject to

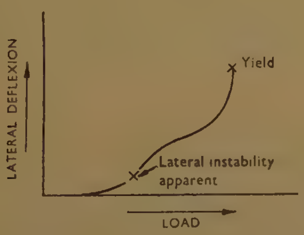


FIG. 7

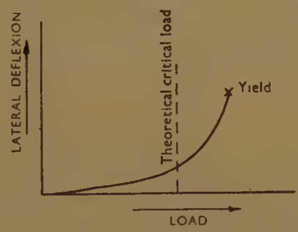


FIG. 8

pure shear, or to shear combined with comparatively low bending stresses. In tests on webs subject to pure bending, Wästlund and Bergmann[9] found that no marked instability of the web was apparent. Deflexions commenced as soon as the loads were applied, and continued to increase until yielding of the web occurred. This phenomenon is discussed below.

EFFECT OF INITIAL ECCENTRICITY

A perfectly flat plate will remain flat until the buckling load is approached; if, however, the plate is not flat, i.e., if it has "initial eccentricity," deflexions from the original profile will occur immediately load is applied to the panel. As the load is increased and approaches the "critical load" at which the buckling of a flat plate would occur, the initial deflexions are magnified, but these are partially restrained by the development of membrane stresses. If the initial deflexions are small compared with the plate thickness, the deflexions before the critical load is reached will be small, and the approach of the critical load marked by rapidly increasing deflexions. As the load is still further increased, the rate of increase of deflexion may be reduced by the membrane stresses (see Fig. 7). If the

initial deflexions are large, then the membrane stresses may be appreciable at loads much less than the critical load. In such a case elastic instability will not be apparent but deflexions will increase gradually from the first application of load until yielding takes place (see Fig. 8). The influence of the initial deflexion on the character of buckling of slender plates is emphasized by Seydel in the summary of his Paper. Wästlund and Bergmann[9] draw attention to this point; out of eleven tests which they carried out on panels in shear, bending, or combined shear and bending only two exhibited signs of instability.

It is reasonable to conclude that in most practical cases the deflexions increase gradually without any marked elastic instability, and that although many girder webs may have high "initial deflexions," these do not cause undesirably large deflexions even as the critical load is approached. In fact, Wästlund and Bergmann point out, in a survey of tests on girder webs with a large variety of specimens, that the maximum deflexions corresponding to the theoretical critical loads for simply supported panels were in no case as large as the plate thickness.

Ultimate failure

A simple definition of ultimate failure is that condition when the plate is unable to withstand further loading. For plate girders this definition has little practical value, since, before this condition is reached, the structure will have become useless or even dangerous. When considering materials such as steel, with a well-defined yield point, it is convenient to take the load at which the plate undergoes large permanent plastic deformation (in its own plane and also laterally) as the *practical* ultimate load. It has been found that local yielding of the material in the plate under combined stresses does not lead to an immediate rapid increase of deflexion in the plate as a whole. The yielding may spread over a considerable area of the plate before failure occurs (this is particularly marked in welded girders owing to locked-up stresses). However, this early local yielding has little or no influence on the *practical* ultimate load.

Comparison of buckling in shear and in bending

It may be useful at this stage to compare the effects of shear and bending on a girder web. Since the web constitutes the primary means of resisting shear, buckling in shear will lead, when further load is applied, to failure of the web. In bending, on the other hand, the web contributes a comparatively small proportion of the total resistance to bending. Hence should the web buckling occur under the action of bending within the elastic range, and further bending be applied, the web will carry little further bending stress, whilst the flanges may resist the increased bending moment. Hence, web buckling due to bending is less important than that

due to shear, and some authorities have suggested lower safety factors for this case.

Permanent set

Any local increase of stress beyond the yield point is, of course, associated with permanent set. As previously stated, local yielding may occur at loads well below the failing load of the plate (particularly in welded girders). It is safe to say that for steel girders this local yielding has no serious effects, and for practical purposes no permanent sets are likely to occur in welded or riveted girders until after the critical buckling load has been exceeded, or the yield stress has been reached over the web as a whole.

Allowable stresses: buckling criterion

In an earlier paragraph, the Authors have proposed rules for checking the strength of web plates. In using these, it is necessary first to determine the stresses at which failure by buckling would occur due to shear and bending acting independently, for a material of constant elastic properties.

The Authors propose that, for this purpose, the failing stresses for failure by buckling be taken as the theoretical instability stresses σ_{cr} and τ_{cr} based on appropriate assumptions of edge-support conditions. This proposal, which would apparently make no reduction in allowable stress for the effects of initial eccentricity, may be justified by reference to three features shown by plates under test :—

(a) Elastic instability of a plate in the elastic range does not constitute failure, which may not occur until considerably higher stresses have been reached.

(b) Deflexions of undesirable magnitude do not occur at the critical load.

(c) Any permanent sets which will have occurred at loads of, say, two-thirds of the critical load will be very small.

Safety factor

It is proposed that the safety factor n introduced to provide a margin of safety against buckling be the same as that implicit in the ratio of the tensile yield stress of the material to its allowable stress in tension. Thus, according to the allowable stresses of the Code of Practice, n would be 1·69 for mild steel to B.S. 15, and 1·84 for high-tensile steel to B.S. 548.

It may be remarked here that the present allowable stresses for webs in shear, according to the Code of Practice, would in many cases show a somewhat lower factor of safety when compared with the calculated buckling stresses.

CONDITIONS OF EDGE SUPPORT IN PRACTICAL CASES AND DESIGN CURVES

It is suggested that the following conditions of edge support be taken for actual cases of edge support :—

(1) Riveted or bolted flanges—clamped edge.
(2) Welded plate flanges—simply supported edge.
(3) Web stiffeners (vertical or horizontal)—simply supported edge. This is suggested since the stiffeners required to satisfy the minimum stiffness requirement (see p. 319) may be very light compared with the flanges.

Consider now a deep girder with riveted flanges sub-divided by, say two horizontal stiffeners, supported by the vertical stiffeners as in Fig. 9

FIG. 9.—HORIZONTALLY STIFFENED GIRDER

For the panels of types A and C, the edges adjacent to the flanges ma be regarded as clamped, and the other edges as simply supported.

In the case where the side adjacent to the flange is the shorter side o the panel in question, the Authors have assumed the clamping effect of th flange on the instability stress in shear to be negligible. The allowabl shear stress can then be found by using the appropriate curve in Fig. 10a These curves are based upon the buckling coefficients given in equation (4) and (5) and Table 1 for simply supported edges using a factor of safet of 1·75, with upper limits equal to the maximum permissible stresses give in the Code of Practice for steels to B.S. 15 and 548 respectively.

In the case where the side adjacent to the flange is the longer side of th panel, the Authors propose to use the curves for permissible shear stre given in Fig. 10b. These have been based on the assumption that th increase in buckling coefficient due to the effect of clamping is half that f both long edges fully clamped (Table 1), using a safety factor of 1·7

Considering a panel of type B (Fig. 9) all edges can be taken simply supported, and the permissible shear stress found from Fig. 10a

For the permissible bending stress for panels of types A, B, and Fig. 10c may be used for all cases where the ratio $\dfrac{\text{length } (b)}{\text{depth}(h_1, h_2, \text{or } h_3)}$ excee 0·75. This has been based upon the buckling coefficients for simp

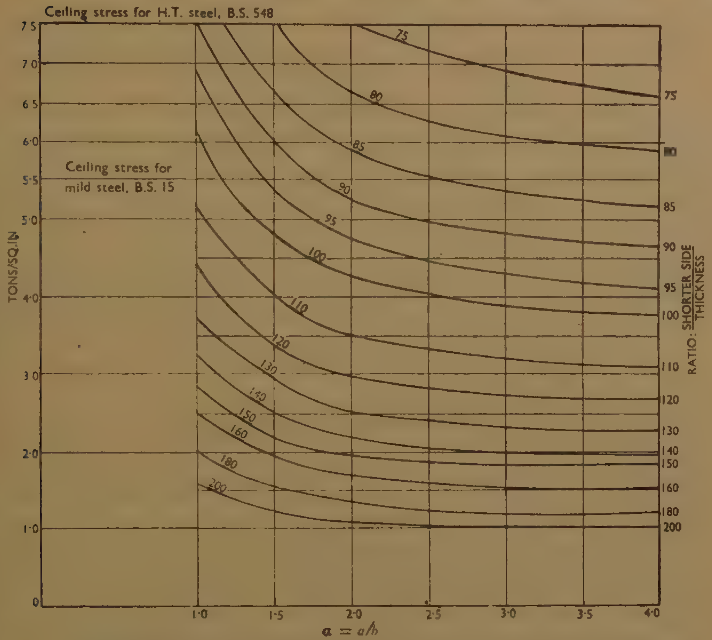


FIG. 10a.—PERMISSIBLE SHEAR STRESSES FOR WEBS (EDGES SIMPLY SUPPORTED)

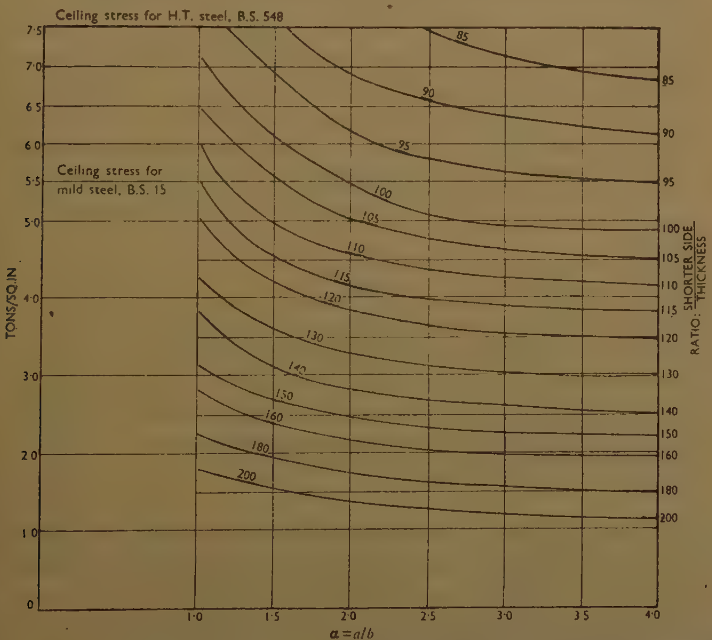


FIG. 10b.—PERMISSIBLE SHEAR STRESSES FOR WEBS (ONE LONGER EDGE CLAMPED)

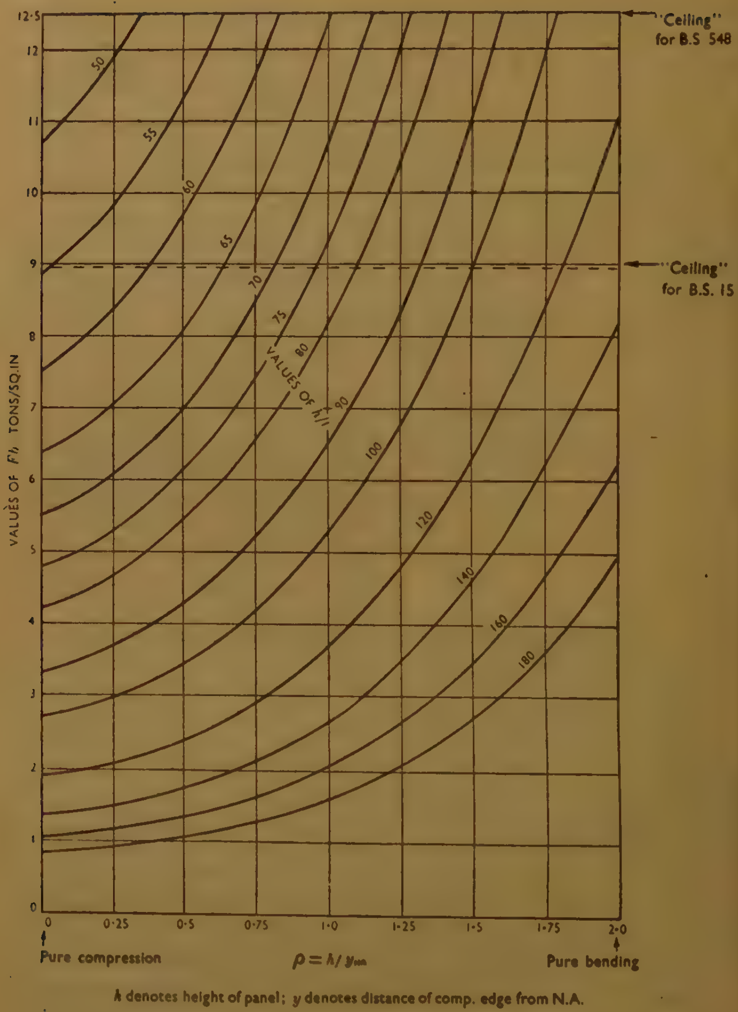


Fig. 10c.—Allowable web stresses in bending
(Side ratio α (length/height) > 0·75)

supported panels given by Timoshenko and shown in Fig. 4, using a safet factor of 1·75. Similar curves may be prepared for length/depth ratic below 0·75. The abscissae of the curves in Fig. 10c represent the rati $\rho = \dfrac{h}{y_{NA}}$, where h denotes panel depth and y_{NA} the distance of the compres

sion edge from the neutral axis. For panels adjacent to the compression flange of a girder of clear web depth d: $\rho = \frac{2h}{d}$.

When considering the theoretical solution for plates in bending, it was pointed out that clamping of the compression edge caused a very considerable increase in the critical stress. Fig. 1 shows that clamping of the compression edge, for girders without horizontal stiffeners, is the principal factor in increasing the buckling coefficient in the ratio $39{\cdot}6/23{\cdot}9 = 1{\cdot}66$. The Authors suggest that a value of 1·50 be adopted when horizontal stiffeners are used. When this value is adopted in conjunction with permissible stresses obtained from Fig. 10c, however, the resulting stress must not exceed the "ceiling" stress for the material in bending.

Theoretical requirements for stiffeners in plate girders

A theoretical approach to the design of stiffeners, given by Timoshenko, is applicable to the study of stiffened plates under shear, compression, or bending. This is based upon the principle of minimum strain energy.

It is found that, for a stiffened plate, two distinct types of buckling can occur. If the flexural rigidity of the stiffener is low, the stiffener and plate will buckle together. With stiffeners of flexural rigidity greater than a certain minimum value, the stiffener remains undeflected, and the plate alone buckles, the buckling load being very nearly that for a plate with rigid supports at the stiffener positions. The minimum flexural rigidity or stiffness of a stiffener, in order that the stiffener will not buckle with the plate, is related to the thickness and dimensions of the web panel. If the stiffness is greater than the minimum value, it may be assumed that the plate is simply supported at the stiffeners. If torsional rigidity of the stiffener is considered, in addition to flexural rigidity, this has a clamping effect on the plate. It is usual to ignore this effect.

Shear

In Fig. 11 curves are shown giving the minimum flexural rigidity of stiffeners for: (1) two adjacent web panels in shear; and (2) three adjacent web panels in shear. In each case b denotes the width, and d the depth of panel. The ordinate γ is the ratio of the minimum flexural rigidity B of the stiffener to D_b (flexural rigidity of the plate itself over the width of the panel). It will be seen that the value for three panels is little more than that for two.

Timoshenko suggests that for practical design of girders with a large number of stiffeners, the necessary minimum stiffness will not be greater than twice that required for a "one stiffener" arrangement of the same depth of web and pitch of stiffeners. The Authors recommend the use of this rule for design purposes, but there are widely conflicting views on

this point. Rockey[1] suggests that 3 times the theoretical value for a "two stiffener" arrangement be used, but, on the other hand, it is stated that the Glenn Martin Aircraft Corporation in America found that, for light-alloy girders, the theoretical value for a "two stiffener" girder is satisfactory for design purposes.

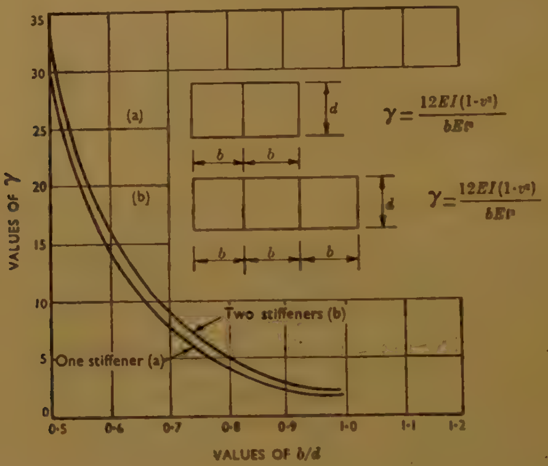


FIG. 11.—MINIMUM FLEXURAL RIGIDITY FOR STIFFNESS IN PURE SHEAR (TIMOSHENKO)

It should be emphasized that a stiffener supporting an initially flat plate does not carry any compressive load until after the buckling load has been surpassed. In actual girders, where the web has an initial eccentricity, membrane stresses develop before the theoretical buckling load is reached, with resulting compression of the stiffeners. However, Professor Turneaure[13] showed, as long ago as 1898, that the compressive stresses in stiffeners are practically zero until buckling or yielding of the web takes place.

COMPARISON OF THEORETICAL AND EMPIRICAL METHODS FOR THE DESIGN OF VERTICAL STIFFENERS

It has been shown in the preceding paragraphs that the essential property of a stiffener in a girder web subject to shear is flexural rigidity. The necessary stiffness may be provided by a section having a relatively small sectional area combined with a comparatively large radius of gyration.

A survey of existing design rules for stiffeners shows that both B.S. 15 and the Code require the stiffeners to be designed as struts, whilst the Code

and B.S. 449 both specify a certain minimum projection from the face of the web. Actually there is no rational relation between such rules and the fundamental property of flexural rigidity. Indian Railway practice is more rational in relating the moment of inertia of the stiffener to the web depth (although the web thickness is not considered). American specifications relate the inertia of the stiffeners to the web depth (and in some cases to the web thickness and stiffener spacing).

In order to illustrate this point, a comparison has been made in Table 3 of the stiffeners required by various regulations already mentioned, using a web thickness of $\frac{1}{2}$ in., web depths of 5 ft and 7 ft 6 in., and stiffener spacings of 2 ft 6 in., 5 ft, and 7 ft 6 in. (The stiffeners are assumed to consist of two or four angles back to back.)

Horizontal stiffeners in girders subject to shear

Whereas B.S. 153 : 1937 and B.S. 449 : 1948 make no reference to the use of horizontal stiffeners in plate girders, approval of the use of such stiffeners to sub-divide the web panels of a plate girder is implicit both in the Code of Practice and the Indian Railway Code. Neither of the latter Codes, however, give any rules for the design of such stiffeners, nor do they suggest any modification of the rules for the design of vertical stiffeners when used in conjunction with horizontal stiffeners. Clearly, if horizontal stiffeners are to be used—and a very strong case can be made for their use—there should be a rational set of rules for the design of girders with horizontal stiffeners. The following approach, which is suggested by the Authors, aims at consistency in the design of the components of plate girders—flanges, web plates, and stiffeners.

The design of flanges is not included in this Paper, and that of web plates has already been dealt with.

In considering the design of stiffeners, the horizontal stiffeners may be assumed to be supported by the vertical stiffeners. The verticals are assumed to remain undeflected until after the point when horizontal stiffeners of minimum stiffness would deflect with the web plate (in the same way that the horizontal stiffeners remain undeflected until after the web plate deflects). The web panels between the vertical stiffeners may be regarded as simply supported panels of the type for which theoretical calculations of minimum flexural rigidity of the stiffener have been made—"one stiffener," "two stiffener," etc. Thus, the horizontal stiffeners of the girder shown in Fig. 9 could be designed by using the curves for a "two stiffener" panel given in Fig. 11, in which $\gamma = B/Db'$ where b' denotes the least distance between horizontal stiffeners. The minimum flexural rigidity of vertical stiffeners required to support the horizontal stiffeners could be found theoretically by an extension of the methods used in the derivation of the curves of Fig. 11. However, so far as the Authors are aware, no general results of such investigations have been published.

TABLE 3

Web	Stiffener spacing	(1) B.S. 153 1937*	(2) Code of Practice	(3) B.S. 449	(4) Indian Railway Code of Practice	(5) A.I.S.C.	(6) Timoshenko	(7) German DIN 4114
5′ 0″ × ½″	2′ 6″ 5′ 0″ 7′ 6″	2 Ls 4″ × 3″ × ½″ 4 Ls 4½″ × 3″ × ⅜″ 4 Ls 6″ × 3″ × ½″	As (1)	2 Ls 4″ × 2½″ × $\frac{5}{16}$″	2 Ls 3″ × 2½″ × ⅜″	2 Ls 2½″ × 2½″ × $\frac{5}{16}$″	2 Ls 4½″ × 3″ × ⅜″ 2 Ls 2½″ × 2½″ × $\frac{5}{16}$″ Nominal	2 Ls 3½″ × 2½″ × ⅜″ Nominal
7′ 6″ × ½″	2′ 6″ 5′ 0″ 7′ 6″	2 Ls 5″ × 3″ × ⅜″ 2 Ls 6″ × 3½″ × ⅝″ 4 Ls 6″ × 3″ × ½″	As (1)	2 Ls 5″ × 2½″ × $\frac{5}{16}$″	2 Ls 5″ × 3″ × ⅜″	2 Ls 3½″ × 3″ × ⅜″	2 Ls 5″ × 3″ × ⅜″ 2 Ls 3½″ × 3″ × ⅜″ 2 Ls 2½″ × 2½″ × $\frac{5}{16}$″	2 Ls 5½″ × 3″ × ⅜″ 2 Ls 3″ × 2½″ × ⅜″ Nominal

Notes:—* Subject to revision.
(1) Based on web shear at 5·5 tons/sq. in.
(2) Based on web shear at 5·5 tons/sq. in, and rule re outstanding leg.
(3) Rule re outstanding leg based on web depth, without reference to stiffener spacing.

They propose, therefore, an approximate method for design purposes, in which the web plate of actual thickness t, acting in conjunction with horizontal stiffeners, is replaced by an "equivalent" web of "effective thickness T" in a girder with vertical stiffeners only. (See Fig. 12.)

T is the least possible thickness of the web of the same girder without horizontal stiffeners, having the same permissible stress as the horizontally stiffened web of the actual girder. If b denotes the shorter side of the panel and F_s the required permissible stress, the maximum value of b/T can be found from Fig. 10a or 10b, and hence T is derived. Having established the value of T, the vertical stiffeners can be designed by means of the curves

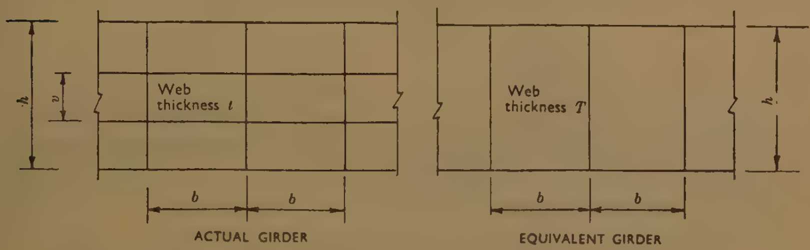


FIG. 12.—EQUIVALENT THICKNESS FOR HORIZONTALLY STIFFENED WEBS

in Fig. 11. This method automatically provides an increase in the strength of the stiffeners over that required for the same thickness of web plate with vertical stiffeners only, except when the permissible stress, even in the absence of horizontal stiffeners, is equal to the ceiling stress.

STIFFENERS FOR WEBS SUBJECT TO BENDING

For webs subject to bending, in which the stiffeners are assumed to be of finite flexural rigidity, buckling may occur in one of two ways, as for stiffened webs in shear. In one case one or more stiffeners will buckle together with the web, whereas in the other case the stiffeners will act as rigid supports during buckling of the web.

VERTICAL STIFFENERS ONLY

Reference to Fig. 1 shows that for a plate of depth h with rigid vertical stiffeners at spacing b, the buckling coefficient in bending for all values of $b/h > 0{\cdot}4$ does not differ by more than 20% from that of an unstiffened plate of the same depth. That is, for this range of side ratios, the vertical stiffeners have little effect in restraining buckling of the plate. Over the same range the flexural rigidity required to provide an effectively rigid stiffener is small. For $b/h < 0{\cdot}4$ however, the stiffeners have considerable

effect in increasing the buckling stress, and the minimum flexural rigidity for effectively rigid stiffeners is correspondingly large.

Comparatively little has been published on the magnitude of the required stiffness. In the current German specification DIN 4114 the following rule is given for the case of a single stiffener separating two adjacent panels :

(1) $0{\cdot}6 < \alpha < 0{\cdot}935$; $\gamma = 6{\cdot}2 - 12{\cdot}7\alpha - 6{\cdot}5\alpha^2$

(2) $\alpha > 0{\cdot}935$; γ is negligible

where $\alpha = b/h$

$$\text{and} \quad \gamma = \frac{\text{flexural rigidity of stiffeners}}{\text{flexural rigidity of width } h \text{ of plate}} = \frac{EI_s}{Dh}$$

Having applied these rules to the design of the vertical stiffeners required to stiffen the girders considered in Table 3, the Authors concluded that in all practical cases the stiffness required for bending was negligible compared with that required for shear.

Horizontal stiffeners only

Many investigators have studied the effect of horizontal stiffeners on the buckling of a web plate in bending. Considering a rigid horizontal stiffener required to divide the web into two panels which buckle simultaneously, such a stiffener must clearly be placed near the compression flange. The appropriate ratio of distance from the compression flange to web depth was given by Massonet [14] (about 1940) as 1/4, but a ratio of 1/5 was suggested by Dubas [15] in 1948 as giving the most economical stiffener. A value of approximately 1/5 was confirmed by the work of Moissieff and Lienhard,[2] who presented curves showing the optimum position for one, two, or three horizontal stiffeners for combinations of shear and bending stress.

By the use of a single horizontal stiffener, providing simple support, at one-fifth of the clear web depth from the compression flange, the minimum buckling coefficient for the web in bending is increased from 23·9 to 129 (cf. Dubas [15]).

The work of Dubas and Massonet covered both the range in which the stiffener behaves as a rigid support to the web plate and that in which it buckles with the web. In the case of continuous horizontal stiffeners, the onset and nature of buckling depends both on the stiffness ratio γ and the area ratio δ, these being respectively those of the flexural rigidity and sectional area of the stiffener to that of a width h of the web plate :

$$\gamma = EI_s/Dh \text{ and } \delta = A_s/ht$$

For discontinuous stiffeners the value of δ may be taken as zero. The curves of Fig. 13 were derived by Dubas and show the variation of the buckling coefficient k in the formula $\sigma_{cr} = k\pi^2 D/h^2 t$ with varying values of γ and side ratio b/h, and are given for values of δ equal to zero and 0·1. (For other values of δ, linear interpolation may be used.)

Massonet published similar information on the buckling of a web with the horizontal stiffener at distance $h/4$ from the compression flange, for which the buckling coefficient corresponding to a rigid stiffener was given as a maximum of 100·8. Hampl[16] published in 1937 the corresponding properties of a web with a horizontal stiffener at mid-height. The buckling coefficient in this case with a rigid stiffener has a value of 36·4.

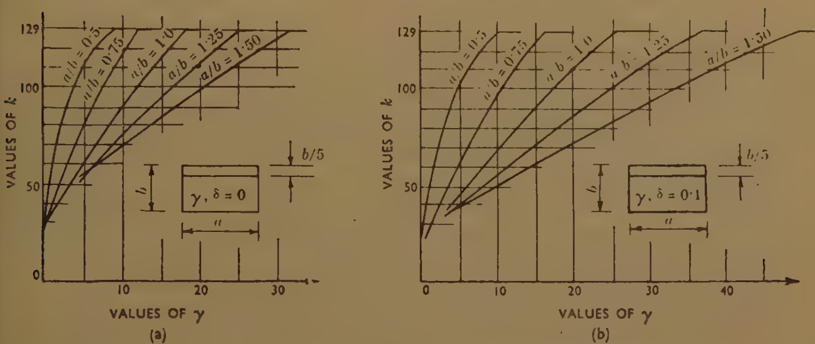


FIG. 13.—BUCKLING OF HORIZONTALLY STIFFENED GIRDERS IN BENDING (DUMAS)

Massonet expressed the view that the most economical stiffener, in the elastic range, was that of just sufficient flexural rigidity to provide an effectively rigid support ("*strictement rigide*"). One factor leading to such a conclusion would be that, theoretically, it is sufficient to provide a stiffener of infinitesimally small area but with high radius of gyration with the necessary moment of inertia (the limiting factor in practice being the thickness of metal). It might be found in any given case that a stiffener of somewhat lower flexural rigidity did not appreciably reduce the buckling stress of the web. The Authors, however, believe that the use of the "strictly stiff" or just-rigid stiffener will generally be justified by the comparative simplicity of this approach, in which the buckling stress of the web is determined only by the stiffener position.

The German regulations (DIN 4114) give values of the stiffness ratio γ for a "strictly stiff" stiffener for three cases, namely mid-height, and $h/4$, or $h/5$ from the compression flange. These regulations also provide for the case where the stiffener and web buckle together, but give the variation of buckling coefficients for varying values of γ only in the case of a stiffener at mid-height.

If a single horizontal stiffener is not placed in any of the positions referred to, the buckling coefficient of the web may be found by considering the individual panels into which the web is divided. It is suggested by the Authors that, for practical purposes, the "strictly stiff" values should be used as given in Table 4 and a safety factor of 2 applied.

TABLE 4

Clear distances from compression flange	Stiffness ratio γ
$s < h/5$	$2 \times \gamma$ for $s = h/5$
$h/5 < s < h/4$	γ for $s = h/5$
$h/4 < s < h/2$	γ for $s = h/4$
$h/2 < s$	γ for $s = h/2$

Considering now the case of two or more horizontal stiffeners, information on the design as well as the positioning of these has been given by Moissieff and Lienhard.[2]

They have drawn curves based on an empirical relation between the area and radius of gyration of practical stiffeners. For a given combination of shear and compressive stresses, the minimum radius of gyration is found, and hence the section of the stiffeners.

In practice it is unlikely that horizontal stiffeners would be placed exactly in accordance with such rules. In most cases it would not be excessively conservative to make all horizontal stiffeners identical, and of a section equal to that for the stiffener nearest the compression flange. The individual web panels may again be treated separately.

COMBINATION OF VERTICAL AND HORIZONTAL STIFFENERS

The buckling of plates in bending when stiffened by both horizontal and vertical stiffeners has been investigated, but published results are confined to a very few simple cases not generally applicable.

In the case of bending without shear, the Authors suggest that the horizontal stiffeners be proportioned as already stated, whilst the vertical stiffeners may be of nominal dimensions. The latter statement can be justified by considering the horizontally stiffened web to be replaced by an equivalent web, having the same buckling stress, with vertical stiffeners only. The stiffness of the latter (for the equivalent web) may be very small.

COMBINED BENDING AND SHEAR

The design of vertical or horizontal stiffeners or combinations thereof has now been considered for the conditions of either pure shear or pure

bending. Little has been published on the subject of the design of stiffeners for webs subject to combined stresses. Milosavljevitch [16] dealt with a simple case of limited application in 1947, but the Paper has certain errors in calculation, and the general nature of the data presented does not appear satisfactory.

The German DIN 4114 also deals only with a few simple cases of combined stress. In these circumstances the Authors propose an approximate rule based on the stiffness requirements for the separate stress conditions as follows :—In a girder subject to combined shear and bending, the necessary flexural rigidities of the horizontal and vertical stiffeners for the separate stress conditions shall be calculated, and added together to find the requisite section.

Stiffeners for webs buckling beyond the elastic range

It has been shown that a web plate may be considered as failing either because of instability or by the yield condition having been reached. The preceding sections on the theoretical requirements for stiffeners have been based upon failure of the web by elastic stability. If, however, failure of the web by yielding takes place, a stiffener designed on the basis of elastic theory will be stiffer than necessary.

To design the stiffeners in the latter case, the Authors propose the use of an "equivalent thickness" method similar to that proposed for the design of vertical stiffeners for horizontally stiffened panels. The following example shows the procedure for webs with vertical stiffeners only (for which it has been shown that shear is the criterion for design). For a web panel where $\alpha = 2$ and $b/t = 60$, the permissible shear stress for a simply supported panel of mild steel to B.S. 15 is found from Fig. 10a to be the "ceiling" stress of 5·5 tons/sq. in., based upon yielding. For the same value of α, the permissible shear stress based on instability would be 5·5 tons/sq. in. when $b/t = 88$ approximately. If the shorter side of the panel is 30 in., then the equivalent thickness $T = \frac{30}{88} = 0{\cdot}34$ in.

Similar methods could be used in the design of horizontal stiffeners both for shear and bending when the permissible stresses are the "ceiling" stresses. In the case of vertical stiffeners supporting horizontal stiffeners, no modification to the design method already proposed is required.

Example of web and stiffener design

Fig. 14a shows a section near the deepest part of the main cantilever girders of the Adhamiyah Bridge, now under construction in Baghdad. The girder is 14 ft 3 in. deep at the section considered, the web plate is $\frac{5}{8}$ in. thick and the flange angles, riveted to the web, are 8 in. $\times$ 8 in. $\times$ 1 in. These angles may be assumed to provide clamping of the web plate. The material is mild steel to a Continental specification equivalent to B.S. 15.

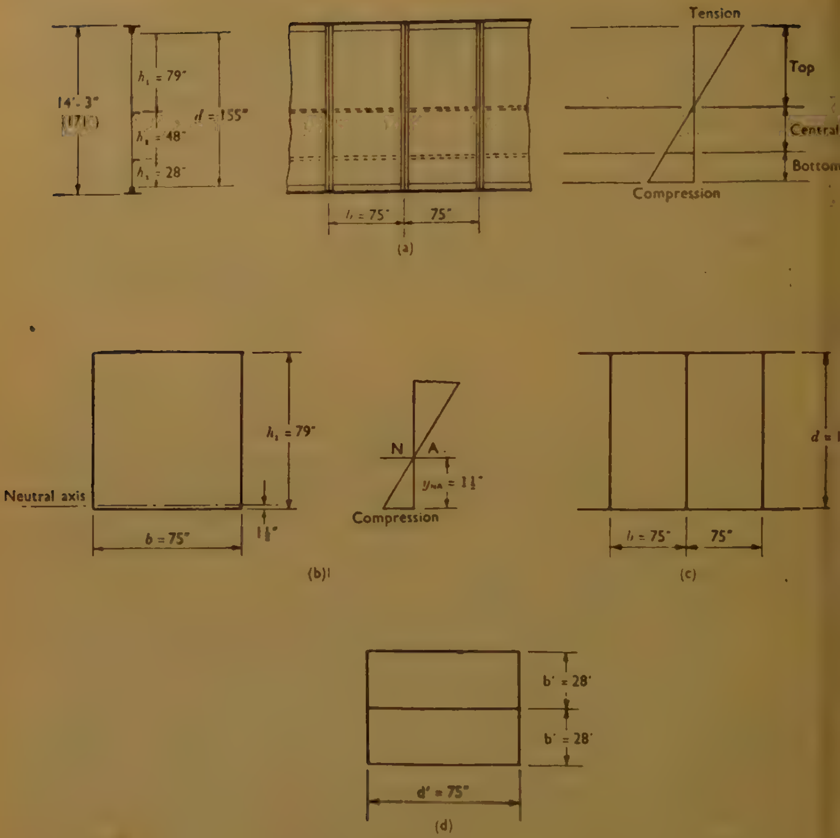


FIG. 14.—SECTION OF MAIN GIRDER, ADHAMIYAH BRIDGE, BAGHDAD

(Panels shown rectangular, using average depth for simplicity in calculation)

Bottom panel

(*a*) *Shear :*

Side ratio (long/short) $= \alpha = b/h_3 = 75/28 = 2{\cdot}68$
Ratio : short-side/thickness $= h_3/t = 44{\cdot}8$

Hence, from Fig. 10b, for one long edge clamped :

$$F_s = 5{\cdot}5 \text{ tons/sq. in.}$$

The shear stress f_s, measured at the toe of the flange angles, is 2·83 tons/sq. in., based on the gross cross-section :

$$\therefore \quad f_s/F_s = 0{\cdot}515$$

(*b*) *Bending :*

Side ratio (length/height) $= \alpha = b/h_3 = 2{\cdot}68$
Ratio : height/thickness $= h_3/t = 44{\cdot}8$

$$\rho = \frac{h}{y_{NA}} = \frac{2h_3}{d} = \frac{2 \times 28}{155} = 0{\cdot}361$$

Hence, from Fig. 10c, $F_b = 9{\cdot}0$ tons/sq. in.
This allowable stress, being the upper limit, applies both to simply supported and clamped edges.

The compressive bending stress f_b, acting concurrently with the shear stress given above, is 6·5 tons/sq. in., based on the gross cross-section :

$$\therefore \quad f_b/F_b = 0{\cdot}722$$

(*c*) *Combined shear and bending :*

Since the permissible stresses for both shear and bending are equal to the upper limits based on yielding, use the "sum of squares" formula :

$$\left(\frac{f_b}{F_b}\right)^2 + \left(\frac{f_s}{F_s}\right)^2 = 0{\cdot}52 + 0{\cdot}29 = 0{\cdot}81 < 1$$

Central panel

(*a*) *Shear :*

Side ratio $\alpha = b/h_2 = 75/48 = 1{\cdot}56$

Ratio : short-side/thickness $= h_2/t = 77$

Hence, from Fig. 10a, $F_s = 5{\cdot}5$ tons/sq. in.
The shear stress f_s is 3·44 tons/sq. in. at the mid-height of the girder :

$$\therefore \quad f_s/F_s = 0{\cdot}625$$

(*b*) *Bending :*

Side ratio (length/height) $= \alpha = b/h_2 = 1{\cdot}56$

Ratio : height/thickness $= h_2/t = 77$

$$\rho = h_2/y_{NA} = 48/49{\cdot}5 = 0{\cdot}97$$

From Fig. 10c, $F_b = 8{\cdot}5$ tons/sq. in.
This figure applies to simply supported edges, and is appropriate to the central panel without any increase for clamping.

Bending stress : $f_b = \dfrac{49{\cdot}5}{77{\cdot}5} \times 6{\cdot}5 = 4{\cdot}15$ tons/sq. in.

$$\therefore \quad f_b/F_b = 0{\cdot}488$$

(*c*) *Combined Bending and Shear :*

Since the panel does not contain the neutral axis, and the permissible stress for bending is less than the "ceiling" stress, use the Parabolic Formula, i.e., formula (14) :

$$\left(\frac{f_b}{F_b}\right) + \left(\frac{f_s}{F_s}\right)^2 = 0{\cdot}49 + 0{\cdot}39 = 0{\cdot}88 < 1$$

Top panel.—Fig. 14b.

(*a*) *Shear :*

Side ratio (long/short) $= h_1/b = 79/75 = 1{\cdot}05$

Ratio : short-side/thickness $= 75/0{\cdot}625 = 120$

Fig. 10a is applicable, since clamping occurs on one short side only :

$$\therefore \quad F_s = 4{\cdot}25 \text{ tons/sq. in.}$$

The mean shear stress f_s on the panel is approximately 3·3 tons/sq. in.

$$\therefore \quad f_s/F_s = 3{\cdot}3/4{\cdot}25 = 0{\cdot}76$$

(*b*) *Bending, and* (*c*) *combined shear and bending :*

The top panel contains the neutral axis, and the applicable rule for combined shear and bending is therefore that for " Case A," i.e., the " sum of squares " rule.

Regarding bending stresses, the rule as expressed in formula (13) refers to the stresses at the compression edge of the panel, i.e., f_b is the actual stress and F_b the permissible stress at the compression edge. If the bending stress on the tension side is greater than that on the compression side, it is necessary to carry out an additional check on the effect of combined stresses on the tension side, using the same rule but inserting the actual and permissible tensile stresses for f_b and F_b respectively.

For the permissible stress at the compression edge, it will be observed that h/y_{NA} is greater than 2 (y_{NA} being the distance of the compression edge from the neutral axis), and therefore Fig. 10c cannot be directly applied. It is reasonable in this case to assume that the allowable compressive stress is that for a panel of depth $2y_{NA}$ subject to pure bending.

In the present example, the value of $2\,y_{NA}$ is only 3 in., and the allowable stress at the compression edge is therefore 9 tons/sq. in. At the same time the actual compressive stress f_b is so small that $(f_b/F_b)^2$ becomes negligible. When combined shear and bending is considered, the " sum of squares " rule as applied to the compression edge is therefore satisfied.

Considering now the combination of tensile and shear stresses near the top flange on the basis of the " sum of squares " rule :

$$\left(\frac{f_b}{F_b}\right)^2 + \left(\frac{f_s}{F_s}\right)^2 = \left(\frac{6{\cdot}5}{9{\cdot}0}\right)^2 + \left(\frac{2{\cdot}83}{5{\cdot}5}\right)^2 = 0{\cdot}52 + 0{\cdot}29 = 0{\cdot}81 < 1$$

The shear and bending stresses f_s and f_b in the web adjacent to the tension (top) flange of the girder are respectively equal in magnitude to the corresponding web stresses at the compression flange. In the present example, the permissible stresses F_s and F_b have been found to have the same values for the web adjacent to the tension and compression flanges, i.e., top and bottom flanges, and the statement of the " sum of squares " rule in the two cases appears identical.

Vertical stiffeners

To proportion the vertical stiffeners it is necessary to consider only the requirements for shear loading and to replace the horizontally stiffened panels by an equivalent web, as in Fig. 14c. The greatest permissible stress in the individual web panels is 5·5 tons/sq. in.

The side ratio (long/short) of the equivalent web between the verticals is $d/b = 155/75 = 2{\cdot}06$. From Fig. 10a, which is appropriate in this case since the shorter edges are clamped, the permissible stress based on instability equals 5·5 tons/sq. in. when $b/T = 88$:

$$\therefore \quad T = b/88 = 75/88 = 0{\cdot}852 \text{ in.}$$

From Fig. 11 the theoretical value of $\gamma = EI_s/Db$ for a one-stiffener panel, when $b/d = 75/155 = 0{\cdot}484$, is found to be 30.

$$\therefore \quad \text{Required } I_s = 2\gamma \cdot \frac{T^3}{12(1-v_2)} \cdot b$$

$$= 2 \times 30 \times \frac{0{\cdot}852^3}{12 \times 0{\cdot}91} \times 75 = 253{\cdot}5 \text{ in}^4$$

Considering four angles each $\frac{5}{8}$ in. thick, the minimum overall depth d over the outstanding legs is now found, since:

$$I_s = 2 \times 1/12 \times 5/8 \times d^3 = 253{\cdot}5$$

$$\therefore \quad d^3 = 253{\cdot}5 \times 9{\cdot}6 = 2{,}440 \text{ in}^3$$

$$\therefore \quad d = 13{\cdot}45 \text{ in.}$$

$\therefore$ Required section of vertical stiffeners is:

4 angles 7 in. $\times$ $3\frac{1}{2}$ in. $\times$ 5/8 in.

Horizontal stiffeners

(*a*) *Shear:*

Since the least vertical distance between the stiffeners or toes of flange angles is 28 in., consider a one-stiffener panel as shown in Fig. 14d.

For the individual panels:

$$\text{Side ratio (long/short)} = \alpha = 75/28 = 2{\cdot}68$$

Since the permissible shear stress is the "ceiling" stress of 5·5 tons/sq. in., it is necessary to determine an equivalent thickness before the minimum required stiffness for shear may be found.

For $\alpha = 2{\cdot}68$, the permissible stress based on buckling equals 5·5 tons/sq. in. when:

$$\frac{\text{short side}}{T} = 85$$

$$\therefore \quad T = 28/85 = 0{\cdot}349 \text{ in.}$$

In order to use Fig. 11, write $b'/d' = 28/75 = 0{\cdot}375$. Extrapolating from the one-stiffener curve of Fig. 11, the theoretical stiffness coefficient γ is found to be approximately 50.

$$\therefore \quad \text{Required } I_s = 2\gamma \cdot \frac{T^3}{12(1 - v^2)} \cdot b'$$

$$= 2 \times 50 \times \frac{0{\cdot}349^3}{12 \times 0{\cdot}91} \times 28 = 11{\cdot}0 \text{ in}^4$$

(*b*) *Bending :*

The required stiffness for bending may be taken as that necessary for the stiffener nearest the compression flange, i.e., the stiffener 28 in. from the bottom flange.

First, the depth of the panel between the stiffener and the compression flange is expressed as a fraction of the clear depth between flange angles :

$$28/155 = 0{\cdot}181$$

This ratio being little less than 0·2, it was decided to use Dubas's curves (Fig. 13), the values of stiffness coefficient being somewhat increased on account of the approximation. In fact, the method adopted is on the safe side since no reduction in effective thickness was introduced on account of the allowable stress in bending being equal to the " ceiling " stress.

In the notation of Fig. 13 :

$$\text{Side ratio } a/b = 75/155 = 0{\cdot}484$$

Using the curves for $\delta = 0$, since the horizontal stiffeners are not made continuous, $\gamma = 10$ say.

Allowing a factor of 2·0 on the theoretical coefficient as for panels in shear :

$$\text{Required } I_s = 2\gamma \cdot \frac{1}{12} \cdot \frac{t^3}{(1 - v^2)} \cdot h$$

$$= 2 \times \frac{10 \times 0{\cdot}625^3}{12 \times 0{\cdot}91} \times 155 = 75{\cdot}6 \text{ in}^4$$

(*c*) *Combined shear and bending :*

Adding together the required stiffnesses for the separate conditions of shear and bending :

Total required $I_s = 11{\cdot}0 + 75{\cdot}6 = 86{\cdot}6 \text{ in}^4$

The section adopted is an 8 in. × $3\frac{1}{2}$ in. × 17·95 lb/ft bulb angle. The relevant moment of inertia of the stiffener is that about the face of the web. This may be expressed as :

I_s provided $= 44{\cdot}82 + 5{\cdot}28 \times 3{\cdot}61^2 = 113{\cdot}6 \text{ in}^4 > 86{\cdot}6 \text{ in}^4$

Summary and conclusions

Consider any girder for which the design bending moment and shear are known. As a first approximation, the web area can be found by dividing the shear by the maximum permissible shear stress. In the case of the flange, however, the area is inversely proportional to the girder depth; by increasing the depth the required flange area may be reduced and the flexural rigidity increased—provided that there is no limitation of the web-depth/thickness ratio. If no such limitation exists, then the optimum depth of the girder may be decided by the following factors: (1) minimum web thickness to resist corrosion and handling stresses; (2) the amount of material required in the form of web stiffeners; and (3) practical considerations, such as construction depth, road or railway gradients, and aesthetic proportions.

The plate-girder type of construction has much to recommend it, both from the point of view of economy of construction and maintenance, and also from that of appearance. Many of the large bridges built in Germany since the late war are of this type,[17] and other examples are the Pont Corneille across the Seine at Rouen,[18] and the new Jersey Turnpike Road Bridges.[19] In all of these bridges it will be found that horizontal (or longitudinal) stiffeners have been used, and the thickness of the web plate reduced thereby. It is probable that the best approach to economic design may be to use high-tensile steel in the flanges and mild steel for web plates, since for the latter the permissible stress may be based upon the elastic constants, which do not vary with the quality of the steel.

If, then, the principles set out in the preceding paragraphs be accepted design methods must be based on some rational foundation in the interests of economy. The Authors have presented this Paper in the hope that it will focus attention of engineers on this subject, which they feel is of increasing importance in the field of competitive design on an international scale.

Acknowledgement

The Authors wish to express their thanks to Messrs Coode & Partners for their kind permission to include in the Paper references to the Baghdad bridges now under construction, in connexion with which the subject matter of the Paper was originally developed.

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The Paper, which was received on the 26th July, 1954, is accompanied by fifteen sheets of diagrams, from which the Figures in the text have been prepared.

CORRESPONDENCE on this Paper should be forwarded to reach the Institution by the 15th September, 1955. Contributions should not exceed 1,200 words.—SEC.

INSTITUTION RESEARCH COMMITTEE

QUALITY OF CONCRETE IN THE FIELD

This report has been prepared by the Committee on the Quality of Concrete in the Field.

W. K. Wallace, C.B.E., M.I.C.E. (Chairman)
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R. H. H. Stanger, A.M.I.C.E.

Table of contents

1. Introduction

On its appointment in 1950, the Committee was given the following Memorandum and Terms of Reference.

Memorandum and Terms of Reference

1. The quality of concrete produced in the field often suffers from lack of appreciation, by both supervisory staff and workmen, of the importance of:

(1) proportioning cement and aggregates,
(2) w/c ratio,
(3) the position of the reinforcement,
(4) cleanliness of reinforcement,
(5) cleanliness of forms,
(6) making of construction joints,
(7) placing of concrete (avoidance of segregation, etc.),
(8) thorough compaction of the concrete.

2. Control by expert supervision, batch mixing, etc., is possible on large contracts and generally leads to fairly good and consistent concrete cube tests, i.e. (1) and (2) are largely solved, but trouble regarding (3) to (7) occurs with distressing frequency. Expert supervision cannot usually be provided on small jobs.

3. However expert supervision may be, it cannot be everywhere at once and bad workmanship is bound to occur as long as the actual workman does not appreciate what good workmanship is.

4. The work of bending and placing reinforcement and proportioning, mixing, and placing concrete is a skilled job and workmen engaged on this work should be trained.

5. The Committee should:

(*a*) satisfy themselves that the above states the case fairly, or amend it appropriately,
(*b*) consider ways and means of training and testing workmen engaged on concrete production,
(*c*) consider any useful aids towards better concrete production, e.g.:
  (i) specification of mixes by weight instead of volume,
  (ii) the introduction of small weigh-batchers, etc., to replace gauge boxes,
  (iii) the use of graded aggregates,
  (iv) the use of mechanical compacting methods,
  (v) setting out the scope of the work of supervision.

In accordance with 5 (*a*) of the above, the Memorandum was considered and the Committee is satisfied that, in the following revised form, it does state the case fairly.

Memorandum

1. The quality of concrete produced in the field often suffers from lack of uniformity in materials and from the lack of appreciation, by both supervisory staff and workmen, of the importance of :

(1) mix design and batching of cement and aggregates,
(2) w/c ratio,
(3) the position of the reinforcement,
(4) cleanliness of reinforcement,
(5) design, workmanship, and cleanliness of forms,
(6) making of construction joints,
(7) placing of concrete (avoidance of segregation, etc.),
(8) thorough compaction of the concrete,
(9) curing.

2. Control by expert supervision, weigh-batching, controlled grading, etc., is possible on large contracts and generally leads to fairly good and consistent concrete as shown by cube tests, i.e., (1) and (2) are largely solved. Cube tests, however, only provide an indication of the potential quality of the concrete as produced at the mixer, and trouble regarding (3) to (9), and particularly (3) and (8), occurs with distressing frequency. The two most obvious faults in concrete work are, insufficient cover to reinforcement due mainly to displacement or inaccurate bending of the bars, and insufficient compaction leading to honeycombing.

3. However expert supervision may be it cannot be everywhere at once and bad workmanship is bound to occur as long as the actual workman does not appreciate the necessity for good workmanship.

4. The work of bending and placing reinforcement and proportioning, mixing, and placing concrete is a job requiring care and workmen engaged upon these tasks should have the need for care impressed on them and should be properly instructed and supervised.

2. Scope

In its deliberations on how better concrete may be produced, the Committee has considered the standards of control possible in the manufacture and placing of concrete and some of the factors involved in choosing the most suitable one for a particular job. The tests, etc., called for by the various standards of control are indicated, and a guide is given for choosing a standard appropriate to the concrete strength required and quantity to be produced. Other points covered are :—

Some of the variations in aggregate which should be guarded against ;
the necessity for good workmanship at all stages of concreting ;
the making, placing, and compaction of concrete ;

the satisfactory bonding of successive pours of concrete made with Portland cement; and

the implications of the specification of concrete by strength.

The recommendations of the Committee for improving the quality of concrete produced in the field are summarized at the end of the report.

3. Control during concrete production

3.1. *General*

The desirable degree of control to be adopted during the production of concrete is governed to some extent by the size of the job. For the larger jobs the value of the cement saved, whilst still maintaining high concrete strength, will make close control at all stages in the production of the finished concrete well worth while. For the smaller jobs, however, it may be cheaper to ensure the required strength by the addition of more cement, but it must be realized that this will not permit standards of workmanship to be relaxed during the placing of the concrete. Within delivery range of suppliers the use of ready-mixed concrete, which offers some of the advantages of a big job, may be worth while. Where high strength is required, equipment and facilities for testing should be available even for small jobs.

3.2. *Degrees of control and grades of concrete*

For the purposes of this report concrete is considered in strength categories (which have been chosen arbitrarily), in each of which the degree of control called for may vary. Four such categories based on specified minimum cube strengths at 28 days have been selected (see Item 3.3) and four standards of control are laid down (see Item 3.4). Considerations affecting the definition of the term "minimum strength" are given in the Appendix.

For a full description of the quality of concrete it is necessary to state the standard of control required to achieve that quality, having regard to the total quantity of concrete to be produced and the minimum cube strength required. Thus, concrete of the highest strength to be produced is described as Grade 1A whilst concrete of medium strength to be produced in small quantities is described as Grade 2B. As an indication of the quality of concrete in each grade, the range of minimum strengths is quoted. Altogether six different grades of concrete are possible (see Table 1); they are 1A, 2A, 2B, 3B, 3C, and 4D.

In general, the higher the quality of concrete the greater is the control required if economical mixes are to be used. The average strengths must be higher than the minimum by an amount which depends on the standard of control. The purpose of control is to improve the uniformity and so enable leaner mixes to be used without detriment to strength. In certain

cases a low-strength concrete may require a high standard of control to ensure the uniformity of other desired properties.

It may not be possible to achieve the higher minimum strengths with concrete of Categories 1 and 2 even by using very rich mixes, unless standard of control A or B is exercised. For concrete of Category 3, it may be possible to obtain the required minimum strength, in the absence of control, merely by increasing the cement content, but this is a wasteful procedure. Further, in those cases where concrete is to be mechanically placed and compacted, uniformity of concrete is demanded by the machines if areas of honeycombing and/or "fatting up" are to be avoided. With Category 3 concrete, therefore, the exercise of control will lead both to more economical mixes and to a better job. For concrete of Category 4 strength is not, in itself, important and may vary within wide limits; rough and ready methods of batching and mixing may then be tolerated, but even in this case such procedure must inevitably lead to waste of cement.

3.3. *Strength category of concrete*

Category 1 concrete is a concrete in which high strength and/or density is required and will normally have a minimum cube compressive strength of over 4,000 lb/sq. in. at 28 days.

Category 2 concrete is one with a good general quality, having a minimum cube strength of between 2,500 and 4,000 lb/sq. in. at 28 days.

TABLE 1

STANDARDS OF CONTROL NEEDED FOR CONCRETE OF VARIOUS STRENGTHS ON WORKS OF VARIOUS SIZES

Concrete strength category	Minimum specified cube strength at 28 days: lb/sq. in	Standards of control for quantities (cu. yd)		
		Large: over 10,000	Medium: 1,000 to 10,000	Small: up to 1,000
1	Over 4,000	A	A	A
2	2,500 to 4,000	A	A or B[1]	B
3	1,000 to 2,500	B[2]	B or C[3]	C
4	Unspecified	D	D	D

Notes: [1] The control would be dependent on the nature of the work.
[2] For large quantities it may be economical to use standard A if the saving in cement justifies it.
[3] The control would depend on the strength specified.

Category 3 concrete is a concrete where high or moderate strength is not a primary requirement, and where impermeability is not absolutely essential. Its minimum cube strength is between 1,000 and 2,500 lb/sq. in. at 28 days.

Category 4 concrete is included to provide for concrete in which quality and strength are unimportant.

3.4. *Control of concrete quality*

3.41. *General.*—In the following requirements for the four standards of control, it is to be noted that, whilst the similarity between the items listed under Standards of Control A and B may lead one to think that the two standards are not very different, this is not the case. The major difference lies in the number and frequency of the tests to be made and reference should be made to Table 2.

In considering the frequency with which tests should be made it is important to bear in mind the necessity of making more frequent tests in the early life of the work. In the case of Control A for example, 12 cubes should be cast each day, in batches of 3, and in the case of Control B, 6 cubes should be cast daily, until the standard of control is established. When the results are satisfactory, the number of tests could be reduced as indicated in Table 2.

3.42. *Standard of Control A.*—When this control is employed, a full mix design is required with tests on the cement and the aggregate and trial mixes. Rigid control of all materials and operations is necessary to produce concrete of the highest quality, and the following operations will normally be found most economical in the long run.

(1) If possible cement from one works only should be used and then, preferably, it should be from a reserved bin. (The Committee realizes that this is not possible at the present time but recommends the re-introduction of bins.)

(2) The coarse aggregate should be obtained in single sizes as defined in B.S. 882 and recombined in the desired proportions at the batching plant.

(3) Regular grading tests of fine and coarse aggregate should be made and corrective measures applied if any drift occurs away from the specified grading.

(4) Materials should be batched by weight.

Cement should be weighed separately to within $\pm$ 2% or measured by the whole bag, if so desired.

The total fine and total coarse aggregates should each be weighed to within $\pm$ 2% after due allowance has been made for their water content.

(5) The weighing devices should be maintained and checked regularly, keeping knife edges clean and bearings well lubricated. The water gauge should be properly maintained and its calibration regularly checked.

(6) It is advantageous to stock-pile the aggregates because it is then

TABLE 2.—REQUIREMENTS FOR THE CONTROL OF CONCRETE QUALITY

Standard of Control	Aggregates				Concrete				Personnel required for testing
	Separation		Tests		Batching		Tests		
	Fine	Coarse	Grading	Moisture determination	Cement	Aggregate	Workability	Strength (see note 2)	
A	One size normally, but two sizes may be required to obtain correct and uniform grading	Single sizes as B.S. 882: e.g., $\frac{3}{8}$ in., $\frac{3}{4}$ in., $1\frac{1}{2}$ in.	After standard has been established, one per week or when variation is suspected	Frequently at first and then as for Standard of Control B [see 3.42 (7)]	By weight or by bag	By weight	Frequently each day if used to control added water	After standard has been established 3 cubes for each 500 cu. yd or part thereof in each day's work	Skilled laboratory technician (full time, or majority of his time)
B	One size	$\frac{3}{4}$ in. to $\frac{3}{16}$ in. graded aggregate plus $1\frac{1}{2}$-in. single-sized aggregate when required	After standard has been established, when variation is suspected	At beginning of each day's work and after moisture content is likely to have changed, e.g., after rain	By weight or by bag	By weight	Frequently each day if used to control added water	After standard has been established 3 cubes each week	Resident Engineer or Assistant (part time)
C	One size Well graded all-in aggregate may also be used	One size	No	No	By weight or by bag	By volume	No	If required	—
D	All-in or unscreened aggregate		No	No	By volume approximately		No	No	—

Notes: 1. The aggregates used in conjunction with Standards of Control A, B, and C should comply with B.S. 882; " Coarse and Fine

easier to maintain the uniformity of the water/cement ratio. Where stock-piles are used, the floor should be clean, the stock-pile should be as large as possible, flat-topped, and drained. It is also recommended that aggregate should not be drawn from the bottom 18 in. or 2 ft of the stock-pile since this is usually much wetter than that above.

(7) Determinations of the moisture content of the aggregates should be made at as frequent intervals as experience shows to be necessary. For the first day or two determinations should be made at the rate of, say, 6 per day. The results would then clearly indicate the frequency of the determinations required subsequently. (Various methods of determining moisture content have been discussed in "Soils, Concrete and Bituminous Materials," published by H.M.S.O. for D.S.I.R. and in Cement and Concrete Association Research Note No. 2, "Review of Methods of Measuring Moisture Content of Aggregates.")

(8) The quantity of water added at the mixer should be adjusted as may be necessary from the results of (7).

(9) Alternatively and preferably, but only when large stock-piles are maintained, the quantity of water added at the mixer should be adjusted from the results of a suitable test on the freshly mixed concrete so as to ensure uniform workability of the appropriate degree for each section of the work. The Slump test is suitable for mixes with a slump of from 2 in. to 6 in. whilst the Compacting Factor test is suitable for drier mixes. (See B.S. 1881.)

(10) Each batch of concrete should be mixed for the specified period.

(11) All transport and handling arrangements should be studied to ensure that any tendency to segregate is corrected, either by modifying the mix proportions or by modifying the arrangements for handling the concrete.

(12) Adequate compaction of the concrete should be ensured, preferably by vibration.

(13) All necessary precautions should be taken to ensure proper curing under any atmospheric conditions.

(14) The appropriate tests should be made on the concrete in accordance with B.S. 1881.

3.43. *Standard of Control B.*—In this standard of control a simple mix design can be used or the proportions can be chosen by experience. The concrete is not so closely controlled as in A, but it still requires care in all the operations if the risk of producing inferior material is to be reduced to practical limits. The following procedure should be adopted :—

(1) The coarse aggregate should be obtained in two sizes when using material of $1\frac{1}{2}$-in. maximum size. In congested sites, however, it may be possible to use only one grade of coarse aggregate, in which case greater attention should be paid to (2).

(2) Regular grading tests of fine and coarse aggregate should be made

and any tendency to depart from specification requirements should be corrected.

(3) All materials should be batched by weight.

Cement should be weighed separately to within $\pm$ 2% or measured by the whole bag, if so desired.

The total fine and total coarse aggregates should each be weighed to within $\pm$ 2% after due allowance has been made for their water content.

(4) The weighing devices should be maintained and checked regularly, keeping knife edges clean and bearings well lubricated. The water gauge should be properly maintained and its calibration regularly checked.

(5) It is advantageous to stock-pile the aggregates because it is then easier to maintain the uniformity of the water/cement ratio. Where stock-piles are used, the floor should be clean, the stock-pile should be as large as possible, flat-topped, and drained. It is also recommended that aggregate should not be drawn from the bottom 18 in. or 2 ft of the stock-pile since this is usually much wetter than that above.

(6) The quantity of water added at the mixer should be adjusted from the results of a suitable test on the freshly mixed concrete so as to ensure uniform workability of the appropriate degree for each section of the work. The Slump test is suitable for mixes with a slump of from 2 in. to 6 in. whilst the Compacting Factor test is suitable for drier mixes. (See B.S. 1881.)

(7) Each batch of concrete should be mixed for the specified period.

(8) All transport and handling arrangements should be studied to ensure that any tendency to segregate is corrected, either by modifying the mix proportions or by modifying the arrangements for handling the concrete.

(9) Adequate compaction of the concrete should be ensured.

(10) All necessary precautions should be taken to ensure proper curing under any atmospheric conditions.

(11) The appropriate tests should be made on the concrete in accordance with B.S. 1881.

3.44. *Standard of Control C.*—The proportions of the mix when this standard of control is considered adequate can be chosen arbitrarily as a result of experience.

The materials can be batched by volume, but allowance should be made for bulking of the fine aggregate.

The gauge boxes or measuring containers should be deep to reduce filling errors. The aggregates need not be separated or supplied in more than two sizes—fine and coarse aggregates. For many purposes all-in aggregate may be used. The control of the water/cement ratio and the appropriate workability of the concrete may be judged by the foreman ganger.

Adequate attention should be given to handling, compaction, and curing.

3.45. *Standard of Control D.*—This standard is included to provide for concrete which is of a quality that does not require any special control in its production.

4. Aggregates

In selecting the aggregate, the Engineer should satisfy himself that the source is suitable for regular supply, and a watch should be maintained to ensure that the particle shape and grading remain reasonably uniform during the progress of the work. For example, in the case of gravels the ratio of the amount of crushed to natural particles may vary as the pit is being worked. This variation will affect the workability of the concrete; the higher the proportion of crushed materials, the lower the workability. Where a partly crushed gravel is used, therefore, attempts should be made to maintain the proportion of crushed to natural materials as constant as possible. Again, when crushed stone is being used there may be a tendency for the aggregate to become increasingly flaky with time due to wear on the crushers and similar factors in production. The greater the degree of flakiness, the more harsh will the mix become and it is thus important to ensure that the proportions of flaky material in the aggregate do not appreciably increase. Where a choice can be made of the shape of the aggregate, it is best to select aggregate of round shape since this will produce concrete of greater workability and/or of greater compressive strength.

5. Control of workmanship

5.1. *General*

Workmanship in all stages of concreting has an important effect on results and should be controlled and supervised by an adequate number of trained and experienced foremen and gangers.

5.2. *Formwork*

The forms, which should be properly designed to withstand all the forces likely to be imposed upon them, should be so constructed that leakage of mortar from the joints will not occur. Before any concrete is placed in the forms, they should be cleaned.

5.3. *Reinforcement*

This should be cleaned free from all loose mill-scale, loose rust, oil, grease, etc. It should be carefully assembled in accordance with the drawings and adequate steps should be taken to maintain it in its correct position.

5.4. *Concrete*

During manufacture the aim should be the production of uniform concrete of the mix specified. During placing, the need is for adequate and uniform compaction without damage to forms or disturbance to reinforcement.

When placing concrete, appropriate tools should be used at each part of the structure to ensure that the concrete is thoroughly compacted. Great care must be taken to prevent reinforcement from being displaced from its correct position; if it is moved it must be replaced immediately.

The concreting foreman must not vary the approved mix or water content without the permission of the Engineer's representative. Throughout each concreting operation he must ensure that the correct mix is delivered unsegregated and without delay.

Concrete of the approved mix design should be sufficiently workable and have the correct mortar content so that it can be compacted fully at all parts of the structure. However, it may occasionally be found, as the work proceeds, that in thin or constricted structural members, the workability must be increased locally in order to effect full compaction, or in parts of the structure where the proportion of reinforcement to concrete is high, that additional mortar is required to fill the voids in the large aggregate. The concreting foreman should do his best to anticipate the need for such variations to the mix so that when they occur the mortar content can be increased without delay. Such local increases of the mortar content, however, should not exceed 10% of the concrete by weight in any single batch and be made only with the approval of the Engineer's representative.

The workability of the concrete must never be altered by the use of additional water or sand alone. When the grading of the aggregate as delivered to the site appears to have varied or the moisture content changed, the laboratory engineer should immediately carry out appropriate tests in order that there may be no delay in advising the Engineer's representative and the concreting foreman of any changes in the mix which it is anticipated will be necessary.

When there is not a site laboratory it should be the responsibility of the concreting foreman to inform the Engineer's representative when he anticipates that a change in the quantity of water added at the mixer will be necessary, so that trial mixes can be made and a revised water quantity determined for the approval of the Engineer's representative.

There are, of course, many other factors outside the actual control and compaction of the concrete which can make or mar good reinforced concrete work. Amongst these attention is drawn to the transporting of the concrete; the planning of the concreting programme to prevent shrinkage cracking and cracking due to the settlement of staging; the correct formation of construction joints by means of temporary vertical stopping-off boards with suitably joggled faces; proper curing of the

concrete especially in hot and dry or very cold weather; and proper attention to the striking of formwork both to avoid local damage to the concrete, or over-stressing of recently poured concrete.

6. Bonding successive lifts of concrete made with Portland cement

Notwithstanding that the workmanship in the actual manufacture and placing of the concrete is satisfactory, concrete work may be defective on account of lack of knowledge or care in the bonding of successive pours. It is important that all supervisory personnel concerned with the execution of concrete work should be instructed in the correct techniques for bonding new concrete to concrete of various ages.

The following general procedure is recommended when bonding concretes, under normal atmospheric conditions.

6.1. *Bonding concrete to that which is not more than 4 hours old*

At the end of each successive lift the "laitance" film and the porous layer immediately below it must be removed before placing the new concrete.

The term "laitance" is applied to the very fine material which, in the presence of an excess of water collects at the top of a lift and forms, on drying, a layer of cement and fine sand, loose in texture, very porous, and susceptible to disintegration if subjected to moisture. Its presence upon the surface of concrete prevents the good adhesion of any new concrete which is to be added.

Where possible it is advisable to fill the forms to a point slightly above the required height, and then strike off the poorer material that collects at the top, before the concrete commences to stiffen up. This procedure should not be delayed beyond a period of 4 hours—since tests have shown that if concrete has stiffened up too much, some of the aggregate below the surface layer may be permanently loosened by the process of removing the top layer, and the possibility of obtaining a good bond with new concrete becomes remote.

Having removed the top layer of concrete, the new concrete must be added immediately. The use of concrete that is too dry will render the efficient bonding of successive layers of concrete very difficult and the concrete at the bottom of each lift will be porous. On the other hand, very wet mixes should be avoided as these result in segregation of the aggregate, and the formation of excessive "laitance," and on setting and hardening the concrete will shrink excessively.

In placing concrete under water the concrete should not be allowed to fall through the water so causing considerable disturbance to the concrete already placed and the washing out of cement from that being deposited, but should be lowered as near as possible to the surface of the concrete already placed before being released.

6.2. *Bonding concrete to that which has been in position for more than 4 hours but not longer than 3 days*

The "laitance" and porous layer should be removed as in 6.1. The surface of the concrete which has now partially hardened, should be brushed with a steel wire brush and thoroughly washed with clean water to remove loose particles, dirt, sawdust, etc., that may have collected in the forms. The surface of the concrete should not be "hacked" unless the concrete has become exceptionally hard. Cement mortar of similar richness to that embodied in the new concrete mix itself, i.e., excluding the large aggregate, and of plastic consistence, should now be applied to the prepared surface while still wet. A layer ½ in. thick would usually suffice. The new concrete should be placed immediately upon the mortar and well punned towards the joint.

6.3. *Bonding concrete to that which is more than 3 days old*

The surface must be chipped or sand-blasted to expose the aggregate and thoroughly rinsed with clean water to remove loose particles. A slurry of neat cement should then be applied. This should be of the consistence of thick cream and may be applied with a brush. The slurry must be well worked into the interstices of the prepared surface. Cement mortar of similar richness to that embodied in the new concrete mix itself and of plastic consistence, should now be applied and this should be followed immediately by the new concrete, which should be well punned towards the joint.

7. Specification of Concrete by Strength

Where it is desired to use concrete of high strength, the Committee suggests that consideration should be given to specifying by strength and when advisable minimum cement content or maximum water/cement ratio, leaving the design of the mix to the contractor. If this is done, the contractor must be allowed additional time for tendering, to enable him to design the mix satisfactorily.

With the above restrictions, the contractor can be left to select the aggregate subject to its compliance with B.S. 882. The purpose of specifying by strength is to take advantage of the contractor's knowledge and skill in securing the required strength with the maximum economy of cost. It might, however, be necessary sometimes for the Engineer to specify maximum cement content so as to obviate excessive shrinkage.

The contractor would be required to satisfy the Engineer that the mi[x] designed by him will give the specified strength during the progress [of] the work. This evidence will be provided by means of preliminary tests carried out in accordance with B.S. 1881. The mean cube strength indi[-]cated by such a series of tests must be equal to the mean works cub[e] strength and would therefore be required to show a margin of strengt[h]

above the specified minimum works cube strength. This margin would be related to the standard of control specified, and for the Standard of Control A should be 1,500 lb/sq. in. and for Standard of Control B, 2,000 lb/sq. in.

It is considered that specification by strength would normally be appropriate for Grade 1 concrete or for Grade 2 concrete of the higher strength range for medium or large jobs, and that Standard of Control A or B should be used to secure the advantages of specification by strength.

The suitability of the contractor's mix design would be checked during the course of the work by means of cube tests made strictly in accordance with B.S. 1881. Where very high strength concrete is being used, these cubes may have to be taken for each day's work and concrete falling below the required standard may have to be rejected.

8. Summary of recommendations

The Committee believes that conscientious supervision can contribute more to the production of good quality concrete than any other single factor. An important step towards achieving conscientious workmanship is to ensure that each man concerned with the production of concrete knows the requirements of those particular parts of the operation on which he is engaged. The Committee believes that efforts should be made to impart this information more widely and, to assist in this, recommends that the series of leaflets entitled " Man on the Job," published by the Cement and Concrete Association, be distributed as widely as possible. A further vital step is to ensure that the workmen are directed and controlled by an adequate number of foremen and gangers properly trained in the technique of reinforced concrete work.

The wider use of small portable weigh-batchers is recommended by the Committee as being conducive to the production of higher quality concrete on small jobs.

The Committee recommends that where circumstances warrant it, consideration should be given to specifying by strength. Since this places the responsibility of producing a suitable mix design on the contractor, his skill and experience is an important factor. It is therefore recommended that tenders for such work should be invited only from contractors selected with this in mind. Additional time for tendering must be allowed to enable the contractors to design the mix satisfactorily.

Where strength, mix, and slump or workability are all specified, care should be taken to ensure that these requirements are not incompatible and do produce a workable mix.

The Committee recommends that mechanical compacting methods should be used in combination with special mix design to achieve economy and certainty of workmanship for high strength concrete. For concrete of normal strength, the Committee is of the opinion that mechanical

compacting methods enable good results to be achieved with a reductio of strain on the supervision of the work.

The Committee recognizes the need for a method of testing the qualit of finished concrete without waiting for the results of cube tests an recommends that investigations should be actively pursued with the objec of developing a test for assessing the quality of finished concrete at ver early ages.

Appendix

CONSIDERATION OF DEFINITION OF MINIMUM STRENGTH

It is often not clear in specifications exactly what is meant by the mini mum strength of the concrete. If the specified minimum strength is con sidered to be an absolute minimum below which no single value will fall, i will be necessary to use a richer and stronger concrete than is reall necessary. By allowing a small number (say not more than 2 to 3% of the results of a large number of tests on cubes to fall below the specifie minimum an economical mix can be designed. In this report the specifie minimum cube strength is taken as the mean strength minus twice th standard deviation* of the results. It should be pointed out that thi definition can strictly only apply after all the cubes have been tested, i.e. after the concreting is finished, whereas the engineer and contractor wis to know at a very early state in the concreting operation that they are, i fact, achieving the desired strength. For works in which a high-qualit uniform concrete is necessary, it is at present the practice in man cases to ask the contractor to supply evidence from trial mixes, made wit samples of the actual materials he proposes to use, that his proposed mi proportions will produce concrete of the specified strength. For concret of this type it is to be expected that the works cubes would give the sam order of strength as that of the trial mixes and the trial mixes shoul therefore give some preliminary evidence that the chosen mix proportion are not likely to be in serious error.

In carrying out the work at the site, 7-day strengths should, in th main, be used to provide the necessary indication of the average strengt and strength variation at 28 days, but a smaller number of 28-day test should be carried out as a check that the actual specification is bein complied with. In considering the making of cubes referred to in the las paragraph of Item 3.41 the normal procedure would then be to test 2 ou of each batch of 3 cubes at 7 days and one at 28 days. At the rate of sets of 3 cubes per day this means that some 24 results should be availabl

* If the differences between the individual results and their average are square and added together and the total then divided by the number of the results minu one, the Standard Deviation is the square root of the resulting figure.

on the 7-day strength after the first 3 days of testing, i.e., 10 days after concreting commences. The results from these 24 cubes should be analysed to determine the average strength, minimum strength, and standard deviation. On the assumption that the 7-day results will be at least 60% of the 28-day strength it will then be possible to make an estimate as to whether the 28-day strengths are likely to achieve the required minimum and, if necessary, adjustments to the mix can be made at that time, i.e., 10 days after the concreting commences. As the work proceeds repeated

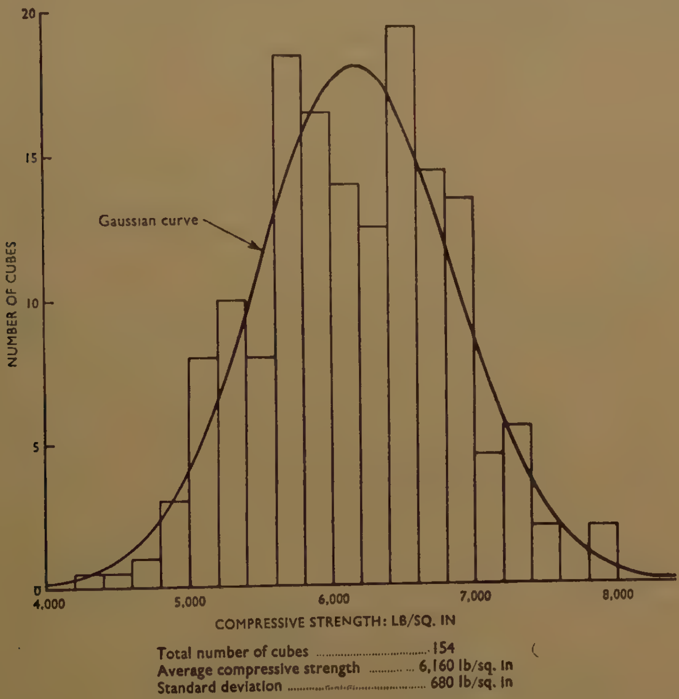

FIG. 1.—HISTOGRAM SHOWING DISTRIBUTION OF CONCRETE CUBE STRENGTHS OBTAINED DURING THE CONSTRUCTION OF A ROAD

This Figure is reproduced by permission of the Director of Road Research.

checks should be made on the results obtained to date so that the possibility that not more than 2 to 3% of cubes shall fall below the minimum is repeatedly under review.

As the cube results are obtained it is convenient to plot them in the form shown in Fig. 1. This is usually done on squared paper using a horizontal scale divided into equal ranges, frequently of 100, 200, or 500 lb/sq. in.

Each cube strength is plotted by filling in a rectangular area in the range in which it falls, these areas being built up into columns. (When a value occurs on the boundary between two ranges it is considered as being half in one zone and half in the next.) This diagram provides an indication of the proportion of cubes which are falling below any specified strength. Thus in Fig. 1 no cube failed below 4,400 lb/sq. in.; two (or $1\frac{1}{3}\%$ of the total) failed below 4,800 lb/sq. in.; and five (or $3\frac{1}{4}\%$) failed below 5,000 lb/sq. in. If the specified minimum had been 4,000 lb/sq. in. and $2\frac{1}{2}\%$ of the cubes had been allowed to fall below that figure the average strength could be reduced from 6,200 lb/sq. in. to about 5,300 lb/sq. in. with a consequent saving of about 20% of the cement.

A REVIEW OF THE METHODS OF TESTING AGGREGATES FOR STRUCTURES OTHER THAN ROADS AND AIRFIELD RUNWAYS

This review has been prepared by the Committee on the Mechanical Properties of Aggregates.

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[1] Deceased March 1953.
[2] Co-opted November 1951.

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PART 3: LIGHTWEIGHT AGGREGATES FOR USE IN CONCRETE

Part 1. General

1.1. *Introduction*

The Committee was originally appointed in 1940 and the only meeting of the Committee prior to the suspension of its work due to the war was held in May of that year. The work of the Committee was resumed in 1946, but, in view of the change of policy of the Institution with regard to research, the method of approach to the problem was reconsidered and the following revised terms of reference were approved by the Council :—

" To study the existing data on the mechanical properties of aggregates, to suggest research where it is considered necessary, and to formulate

methods of testing the mechanical properties of aggregates and how they can best be used in practice which would correlate the results obtained thereby with the performance of aggregates when used in combination with cement and bituminous materials."

In 1950, "A Review of the Methods of Testing Aggregates for Roads"* was prepared by the Committee. In the preparation of that Review the Committee considered the use of aggregates in relation to all branches of engineering construction, but as it became increasingly clear that the problems associated with the use of aggregates for road construction and maintenance were more complex than in any other branches of civil engineering, it was confined to the use of aggregates in roads, and certain tests for properties of aggregates which were not important for roads were omitted. This Review deals with those tests, but for the sake of completeness includes also some of the tests reviewed in the earlier Report. It does not include consideration of the special properties of aggregates required for airfield runways.

The Committee has found it difficult to define with exactness the terms "mechanical properties" and "aggregates," and, as in "A Review of the Methods of Testing Aggregates for Roads," both terms have in this Report been broadly interpreted.

1.2. *The significant properties of aggregates*

The following is a list of properties of aggregates which may be responsible for their performance in structures. Whilst all these properties may be significant, their relative importance depends upon the purpose for which the aggregates are to be used and the nature of the cementitious material.

- 1.201. Particle shape and surface texture
- 1.202. Water absorption, specific gravity, and unit weight
- 1.203. Grading
- 1.204. Uniformity
- 1.205. Petrological characteristics
- 1.206. Mechanical strength
  - 1.2061. Resistance to impact
  - 1.2062. Resistance to crushing
- 1.207. Impurities
  - 1.2071. Clay, fine silt, and fine dust
  - 1.2072. Organic materials
  - 1.2073. Water soluble salts
- 1.208. Behaviour at high temperatures
  - 1.2081. Physical stability and thermal expansion
  - 1.2082. Chemical stability under fire

* J. Instn Civ. Engrs, vol. 35 (1950–51), p. 211 (Jan. 1951).

1.209. Chemical stability
1.2091. Reactions with cement
1.2092. Oxidation or hydration of aggregate, or impurities
1.210. Resistance to weathering

1.3. *Factors affecting the performance of aggregates*

The performance of aggregates in a structure may depend, *inter alia*, upon the following factors, which are independent of the properties of the aggregate itself:

(*a*) the methods used in the production or winning of the aggregates;
(*b*) the methods of mixing with the cementitious material;
(*c*) the method of transport to the site, storage, handling, placing, compacting, and finishing;
(*d*) the degree of control in the placing, compacting, and finishing;
(*e*) the type of usage;
(*f*) the climatic conditions.

Part 2. Aggregates other than lightweight for use in concrete

2.1. *General considerations*

The qualities required of such aggregates are that they shall be cheap and plentiful, mechanically strong, chemically stable, and inert. There are other desirable qualities some of which are considered below.

2.11. *Behaviour of aggregates at high temperatures*

The following classification of concrete aggregates in respect of fire resistance is conveniently based on their behaviour at high temperatures *:

Class 1(*a*) foamed slag, pumice. (These are lightweight aggregates—see Part 3 of this review.)
1(*b*) blast-furnace slag, crushed brick and burnt clay products, well-burnt clinker, crushed limestone.
Class 2 siliceous aggregates generally, e.g., flint, gravel, granite, and all crushed natural stones other than limestone.

Those in Class 1, with the exception of limestone, are virtually unaffected by temperatures up to 1,000°C. Limestone which calcines at about 700°C is liable to disintegrate upon subsequent exposure to air.

Class 2 aggregates, which are generally siliceous, may be liable to spalling, and in some cases shattering at temperatures around 570°C, the shattering being due either to dehydration of the mineral or inversion of quartz from one form to another. The change in colour of the aggregates on heating is generally due to changes in the structure of the iron compounds present in the aggregate.

* "Fire Gradings of Buildings," Post-War Building Studies No. 2, Appendix VI, H.M.S.O., 1946.

This classification takes into account only the fire resistance of the aggregates, and where the aggregates are to be used for structural concrete their characteristics in respect of concrete must be considered.

2.12. *Thermal expansion*

The coefficient of thermal expansion for normal aggregates varies between $2{\cdot}4 \times 10^{-6}/°F$ for limestones and $6{\cdot}4 \times 10^{-6}/°F$ for quartzite; igneous rocks lie between these limits. For information on the thermal expansion of concrete reference should be made to the Paper by Bonnell and Harper.*

2.13. *Chemical stability*

2.131. *Reactions with cement*

It has been found that certain aggregates containing opaline silica, highly siliceous glasses, or other reactive silica may interact with alkalis (NaOH or KOH) present in cement, to cause expansion and disruption of concrete. Such reaction usually occurs with cement high in alkali, but may occur with low alkali cement if the reactive material and alkali are present in the proportions necessary for reaction.

In work carried out at the Building Research Station none of the naturally occurring British aggregates normally used have so far been found to cause harmful expansion at normal temperatures (20°C).† However, where extensive new work is to be undertaken, particularly abroad, using aggregates for which there is no previous experience, tests should preferably be carried out to determine if there is risk of expansive reaction. (See also recommendation 4.107.)

Another possible type of reaction is that of iron-containing aggregates with free lime produced by hydration of cement. Work at the Building Research Station indicates, however, that this is not likely to have any harmful effect.

A third type of reaction is that sulphate-containing aggregates may react harmfully with the calcium aluminate constituents of cements.‡

2.132. *Oxidation or hydration of aggregate*

A possible danger exists of oxidation, perhaps accompanied by hydration, of certain aggregates, giving rise to expansion which may be detri-

* D. G. R. Bonnell and F. C. Harper, "The Thermal Expansion of Concrete." J. Instn Civ. Engrs, vol. 33, p. 320 (Feb. 1950).

† F. E. Jones, National Bldg. Studies Res. Papers 14 and 15, H.M.S.O., 1952.
F. E. Jones and R. D. Tarleton, National Bldg Studies Res. Paper 17, H.M.S.O., 1952.
F. E. Jones, "Alkali-Aggregate Interaction in Concrete," *Chem. and Ind.* 1953, pp. 1375–83.

‡ Institution of Civil Engineers, Committee on the Soil Corrosion of Metals and Cement Products, "Report from the Sub-Committee on Environment." J. Instn Civ. Engrs, vol. 14 (1939–40), p. 392 (June 1940).

mental to the bond between aggregate and cement. Such a mechanism has been advanced for failures of concrete made with certain types of altered dolerites. The evidence, however, does not appear to be conclusive and work on concrete disintegration at the Building Research Station has suggested that other factors may be more important. It is reasonable to require that an aggregate should be stable against oxidation or hydration, but there appears as yet no adequate ground for requiring a specific test, possible exceptions being certain flint gravels containing pyrites or marcasite. The latter minerals produce very unsightly iron staining on the exposed surface of concrete.

2.133. *General chemical attack on concrete*

The disintegration of concrete in contact with aggressive chemical agents is not generally due to breakdown of the aggregate since normally the aggregate is an inert material, the exception being principally limestone and aggregates containing calcium carbonate. Normal Portland cement on hydration forms products which are prone to attack by weak acids or by certain salts in solution in water. The resistance to attack is so much dependent upon the permeability of the concrete, which in turn depends upon the mix and the workmanship, that hard and fast rules regarding the behaviour of "concrete" cannot be laid down. The type of usage which the concrete has to withstand is also an important factor affecting durability under chemical attack, so that in conditions where the reaction products can be removed either by leaching or by abrasion, attack is likely to be accelerated. The leaching effect of pure water passing through improperly compacted concrete may be sufficient to impair the quality of the material. Table 1 is included to give an indication only of the range of materials which are aggressive to matured concrete.

Table 1 indicates that exposed concrete or mortar is liable to attack under many industrial circumstances, and there is considerable practical experience of the effects of many agents. Thus concrete floors in buildings processing sugars, animal fats, meat, seed oils, soap, and milk may be attacked. Leakage or spillage of calcium chloride brine in refrigeration plants has been known to cause disintegration.

The attack of concrete in foundations by ground-waters containing sulphates in solution, and of marine structures by the magnesium sulphate present in sea water, is all too common. The aggressive action of waters draining from coal and coke storage bunkers has been similarly attributed to the formation of sulphates and free sulphuric acid.

The attack of concrete in situations where vapours of an acidic nature are present is well known, e.g., in railway tunnels and in chimneys. In certain industrial operations attack may occur when conditions are humid.

Allied to the latter type of attack is the deterioration occurring in some instances in sewers where conditions of temperature and time of retention are favourable to the production of hydrogen sulphide by bacteria in the

TABLE 1

Material	Effect
Mineral oils : (petrol, fuel oil, and petroleum distillates in general)	No action
Vegetables and animal oils and fats	Attack
Coal tar oil	Attack
Organic acids :	
Lactic acid	Attack
Acetic acid	Attack
Oxalic Acid	Slight action
Oleic, stearic, palmitic acids	Attack
Inorganic acids	Attack
Alkali hydroxides	Attack
Sulphates : (e.g., sodium, magnesium, calcium, ammonium)	Attack
Chlorides :	
Sodium,* potassium	No action
Calcium (strong solutions)	Attack
Nitrates :	
Sodium, potassium	No action
Ammonium	Some attack
Sugar	Attack

* If the salt contains sodium sulphate it is liable to attack.

sewage. This gas, evolved into the sewer air, undergoes oxidation, resulting finally in the production of sulphuric acid which rapidly attacks the soffit of the sewer.

For further information on chemical attack on concrete reference may be made to :—

F. M. Lea and C. H. Desch, " The Chemistry of Cement and Concrete." Arnold, London, 1935.

Institution of Civil Engineers, Committee on the Deterioration of Structures in Sea-water, 15th Report, p. 107, 1935.

A. Kleinlogel, " Einflusse auf Beton." Ernst, Berlin, 1930.

American Concrete Institute, Joint Committee on Standard Specifications for Concrete and Reinforced Concrete, " Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete," 1940.

2.14. *The weathering of concrete*

The density and impermeability of concrete are the main factors in determining its resistance to weathering. Weathering may be due to

industrial atmospheres, sulphate efflorescence, or to frost action. Industrial atmospheres which contain sulphuric acid will rapidly attack porous concretes and cause expansion or spalling. Although the general cause of frost disintegration is known to be the expansive force developed when water turns to ice, the relation between the character of the pore structure of a material such as concrete and its resistance to frost is complicated.

2.2. *Tests on aggregates*

Table 2 shows the principal tests for determining the mechanical properties of aggregates :—

TABLE 2

Significant properties	Reference to tests
2.201. Particle shape and surface texture	
2.2011. Description	B.S. 812 : 1951, clause 10[1]
2.2012. Flakiness	B.S. 812 : 1951, clause 15
2.2013. Elongation	B.S. 812 : 1951, clause 15
2.2014. Roundness or angularity	(See below)[2]
2.202. Water absorption, specific gravity, and unit weight	B.S. 812 : 1951, clauses 16, 17, 19 ; and J. App. Chem., 1953 : 3 (3) 110–7[3]
2.203. Grading	B.S. 812 : 1951, clause 11
2.204. Uniformity	No generally accepted standard test
2.205. Petrological characteristics	B.S. 812 : 1951, clause 9
2.206. Mechanical strength	
2.2061. Resistance to impact	B.S. 812 : 1951, clause 23
2.2062. Resistance to crushing	B.S. 812 : 1951, clause 25
2.207. Impurities	
2.2071. Clay, fine silt, and fine dust	B.S. 812 : 1951, clauses 12, 13, 14
2.2072. Organic materials	B.S. 812 : 1951, clause 22
2.2073. Water soluble salts	No generally accepted standard test
2.208. Behaviour at high temperatures	,, ,,
2.2081. Physical stability and thermal expansion	,, ,,
2.2082. Chemical stability under fire	,, ,,
2.209. Chemical stability	,, ,,
2.2091. Reactions with cement	,, ,,
2.2092. Oxidation or hydration of aggregate, or impurities	,, ,,
2.210. Resistance to weathering	,, ,,

[1] B.S. 812 : 1951, " Sampling and Testing of Mineral Aggregates, Sands and Fillers."

[2] F. A. Shergold, " The percentage voids in compacted gravel as a measure of its angularity," *Mag. Conc. Res.*, 1953, vol. 5, No. 13, p. 3.

[3] F. A. Shergold. " The test for the apparent specific gravity and absorption of coarse aggregate," *J. App. Chem.*, 1953, vol. 3, No. 3, p. 110.

2.3. *The significant properties of concrete*

The Committee has found it necessary to consider tests on concrete

because there are properties of concrete which depend on the aggregates used but cannot be wholly determined by tests on the aggregates alone. These properties are :—

2.31. Strength
2.32. Unit weight
2.33. Thermal conductivity
2.34. Drying shrinkage and moisture movement
2.35. Resistance to weathering
2.36. Behaviour at high temperatures
2.37. Resistance to chemical attack
2.38. Resistance to wear

2.4. *Tests on concrete*

Table 3 shows the principal tests for determining the properties of concrete which are dependent on the aggregates used :—

TABLE 3

Significant properties	Reference to tests
2.41. Strength	B.S. 1881 : 1952, Part 8[1]
2.42. Unit weight	B.S. 1881 : 1952, Part 5
2.43. Thermal conductivity	D.S.I.R. Report[2]
2.44. Drying shrinkage and moisture movement	B.S. 1881 : 1952, Part 16
2.45. Resistance to weathering	No generally accepted standard test
2.46. Behaviour at high temperatures	,, ,,
2.47. Resistance to chemical attack	,, ,,
2.48. Resistance to wear	B.S. 368 : 1936[3]

[1] B.S. 1881 : 1952, "Methods of Testing Concrete."
[2] E. Griffiths, "The Measurement of the Thermal Conductivity of Materials used in Building Construction," J. Instn H. & V. Engrs, vol. 10 (1942), p. 106.
[3] B.S. 368 : 1936, "Pre-cast Concrete Flags."

2.5. *Tests on aggregates and concrete*

Some of the tests referred to in Sections 2.2 and 2.4 are not considered to be wholly satisfactory. The enquiry also revealed that there are at present no suitable tests to determine some of the properties which may be significant.

The Committee has made a number of recommendations for the development of existing tests and the formulation of new tests where such a course appears desirable.

The Committee has noted the development of ultrasonic testing of concrete and hopes that this may further develop, so that indestructive testing can become more general. All tests on concrete which are carried out to assess the property of aggregate should be made on test pieces prepared by a standard method.

The procedures for carrying out some of the tests listed in Sections 2.2 and 2.4 are summarized in Appendices A and B.

2.6. *Discussion of tests on aggregates*

2.601. *Particle shape and surface texture*

The British Standard methods for describing (A–1.1) and for measuring the flakiness and elongation (A–1.2) of particle shapes are acceptable. The Committee considers, however, that in view of the increasing importance attached to surface texture as a factor affecting the strength of concrete, some quantitative measure of this property is desirable in addition to the qualitative description given in B.S. 812 : 1951. (See also recommendation 4.101.)

The Committee considers that the test for roundness or angularity developed at the Road Research Laboratory and described in Appendix A (A–1.3) is the most satisfactory test for this purpose brought to its notice. (See also recommendation 4.102.)

2.602. *Water absorption, specific gravity, and unit weight*

The Committee considers that the British Standard tests for water absorption, specific gravity, and unit weight (A–2) are generally satisfactory, except that the procedure for coarse aggregates should be modified in the light of recent research referred to in the footnote to section 2.202. (See also recommendation 4.103.)

2.603. *Grading.*—The Committee considers the British Standard test for sieve analysis (A–3) to be satisfactory.

2.604. *Uniformity.*—There is no test suitable for standardization for the assessment of the uniformity of aggregates in bulk. The Committee considers that such a test is desirable. (See also recommendation 4.104.)

2.605. *Petrological characteristics.*—The Committee feels that engineers should have information on the geological identification of aggregates and their allocation to the appropriate Group classification (A–4).

2.606. *Mechanical strength*

2.6061. *Resistance to impact.*—The Committee is of the opinion that resistance of an aggregate to impact is best determined by an impact test of the Stewart type (A–5.1).

2.6062. *Resistance to crushing.*—The British Standard aggregate crushing test is regarded as satisfactory (A–5.2).

2.607. *Impurities.*—The Committee considers that research is required to determine what impurities in aggregates are harmful, and if present, up to what limits they are permissible, and to devise suitable tests.

The British Standard tests for clay, fine silt, and fine dust,

and the approximate test for organic impurities (A–6.1 anc A–6.2 respectively) may be misleading. (See also recommendation 4.105.)

2.608. *Behaviour at high temperatures.*—There is no standard tes for resistance to fire which is applicable to an aggregate or to a concrete made from that aggregate. Fire resistance test are made upon unit structural elements representative of botl the type of construction and the materials used. Indeed, th term " fire resistance " is restricted by definition in B.S. 476* to a property of a structural element when tested in a prescribe manner. The Committee considers that correlation of th properties of aggregates and concretes with the behaviour o structural elements formed therefrom under the standard fir resistance tests, to be a subject upon which research is urgentl required with a view to simplifying fire resistance testing (See also recommendation 4.106.) It is considered that such test would probably be necessary only where there was a risk o high temperatures in the structure.

2.609. *Chemical stability : tests for alkali-reactive aggregates*

Petrographic examination is the most direct approach for determinin the presence of possibly reactive material. It requires, however, the se vices of an expert petrographer who is also familiar with the specifi problem. A negative result does not necessarily mean that the whole c the aggregate is free from the reactive material.

Of the test methods which have been advocated, short period chemica tests (A–7) should be regarded as a guide rather than a positive tes since indications of reactivity may not necessarily indicate harmful expa sion in concrete. For example, flint aggregate which is found to be re active in the rapid chemical test is not expansively reactive at 20°C. Th test should therefore be regarded as an acceptance test only where no reactivity is shown. It should not be used as a rejection test, unless co firmed by the expansion-bar test (A–7). The direct determination c the expansion of a bar of mortar or concrete incorporating the aggrega under test is more reliable, though it has the disadvantage of requirin a period of at least 6 months and possibly up to 1 year. The Committe recommends that further research be undertaken to develop a reliable an rapid method of testing for the presence of reactive silica liable to cau expansion. (See also recommendation 4.108.)

2.610. *Resistance to weathering.*—No generally accepted standa test exists at present. The necessity for such a test in Brita may not be very great when good quality aggregates are use

* B.S. 476 : 1932, " Definitions for Fire-Resistance, Incombustibility and No Inflammability of Building Materials and Structures (Including Methods of Test)."

but the test might be important in determining the suitability of low-grade aggregates.

2.7. *Discussion of tests on concrete*

2.71. *Strength.*—The British Standard test for strength of concrete (B–1) is acceptable.

2.72. *Unit weight.*—The British Standard method of determining the unit weight is satisfactory (B–2).

2.73. *Thermal conductivity.*—The methods used by the National Physical Laboratory described by Dr E. Griffiths (B–3) are considered satisfactory. This test cannot, however, be carried out except at laboratories which have the special apparatus and personnel conversant with the technique.

2.74. *Drying shrinkage and moisture movement.*—The British Standard test is acceptable (B–4).

2.75. *Resistance to weathering.*—Existing test methods rely on a comparatively rapid succession of freezing and thawing cycles, usually carried out on a saturated specimen.

Whilst such tests give some indication of the behaviour of materials in practice there are notable exceptions and certain types of specimen which fail under the accelerated tests are comparatively unaffected by natural weathering or tests in which the freezing cycles are separated by a long interval.

A suitable form of test has still therefore to be devised. (See also recommendation 4.109.)

2.76. *Behaviour at high temperatures.*—As stated in Section 2.11 the classification of aggregates in respect of fire resistance takes into account only the resistance of the aggregate. (See also Item 2.608.)

A test for aggregates to be used in structural concrete is at present best carried out as a compressive strength test on the concrete before and after calcining under definite conditions, combined with visual inspection during and after heating.

In the case of a limestone aggregate, a further test, such as immersion in water might be necessary.

2.77. *Resistance to chemical attack.*—There are no standard tests for the resistance of concrete to chemical attack and in view of the great variety of chemicals to be considered, it is difficult to visualize any such tests being formulated. For information on materials liable to attack concrete reference should be made to Item 2.133.

2.78. *Resistance to wear.*—The Committee is of the opinion that the test described in B.S. 368 (B–5) is suitable for concrete flags, but it is recommended that the dust should be continuously removed during the course of the test.

It has not yet been possible to devise a test for resistance t wear that would be suitable for concrete in general and it is recom mended that research should be undertaken to formulate such test. (See also recommendation 4.110.)

2.8. *Sampling*

It is important that sampling should be conducted in a standar manner, and the Committee considers that the British Standard samplin procedures are acceptable. Summaries of the methods are given i Appendix C.

When sampling aggregates the Committee considers it desirable t encourage the use of sample-splitters. A common type—the "riffl box"—consists of a number of narrow chutes discharging alternatively o opposite sides of the box; the sample is poured evenly over the top of th box and is thus split into two representative halves, which are collecte in boxes placed under the chutes. The operation is repeated until th desired size of sample is obtained. Two sizes of riffle are normall required—one for coarse aggregate and the other for fine aggregate For other types of sample-splitters, reference should be made to B.S. 1017 Appendix A.*

PART 3. LIGHTWEIGHT AGGREGATES FOR USE IN CONCRETE

3.1. *General considerations*

During recent years there has been an increasing demand for lightweigh concrete and there is a considerable amount of literature available on th subject.† Its lower unit weight compared with the more traditiona materials offers opportunities in some cases for the reduction of initia costs and greater productivity in construction. Where it is used for ther mal insulation, its lower thermal conductivity may lead to a reduction i heating costs.

3.2. *Types of lightweight aggregates*

A number of different lightweight aggregates, which vary widely i their physical properties and give rise to similar variations in the propertie of the concrete they produce are available. For convenience they are her classified into four types.

Type A. Naturally occurring
Type B. Furnace clinker
Type C. Artificially expanded materials (other than Type D)
Type D. Very lightweight aggregates (Artificially expanded)

* B.S. 1017 : 1942, "Sampling of Coal and Coke."
† See Bibliography, Item 3.9.

The examples of these types which are available in Great Britain and which have been considered in preparing this review are given below :—

Type A. Pumice : a naturally expanded volcanic glass.
Type B. Clinker : well-burnt furnace residues which have been fused or sintered into lumps.
Type C. Foamed slag : a cellular material manufactured by treating molten blast-furnace slag with sufficient water or other suitable medium to produce a cellular dry product which is crushed and graded as required. It consists chiefly of alumino-silicates of lime and magnesia in a glassy, partly crystalline, or crystalline condition.
Expanded shales and clays : crushed or preformed shales or clays expanded by heating.
Type D. Expanded vermiculite : a mica-like material expanded by heating.

3.3. *Tests on lightweight aggregates*

Certain tests are available for assessing some of the more significant properties of particular lightweight aggregates and reference to them is made in Table 4. Owing to their different physical and chemical nature, a test developed for one aggregate may not be applicable to another.

A property of an aggregate which may be considered significant may be impossible to test satisfactorily except by making specimens of concrete and inferring that property from the behaviour of the concrete.

3.4. *Significant properties of lightweight concrete*

The Committee has found it necessary in certain cases to consider tests on lightweight concrete because there are properties of the concrete which depend on the aggregates used but which cannot be wholly determined by tests on aggregates alone. Such properties are :—

3.41. Unit weight
3.42. Strength
3.43. Drying shrinkage and moisture movement
3.44. Moisture absorption and retention
3.45. Thermal conductivity
3.46. Behaviour at high temperatures
3.47. Resistance to weathering

The resistance to weathering is only significant if the concrete is to be used for external work.

3.5. *Tests on lightweight concrete*

There are two aspects of concrete tests. First, the suitability of an aggregate for a particular purpose may be fully assessed by testing concrete

TABLE 4

Significant property	Applicable to aggregate type	Tests
3.301. Unit weight	Foamed slag	B.S. 877[2]
	Others	N.S.[1]
3.302. Grading	All	N.S.
3.303. Particle shape and surface characteristics	All	N.S.
3.304. Water absorption	All	N.S.
3.305. Strength and compressibility	All	N.S.
3.306. Impurities		
3.3061. Sulphate	Furnace clinker	B.S. 1165[3]
	Foamed slag	B.S. 877
	Others	N.S.
3.3062. Combustible material	Furnace clinker	B.S. 1165
	Foamed slag	B.S. 877
	Others	N.S.
3.3063. Heavy impurities	Foamed slag	B.S. 877
	Others	N.S.
3.3064. Silt, clay, etc.	All	N.S.
3.3065. Free lime	All	N.S.
3.3066. Oxidizable iron compounds	All	N.S.
3.307. Uniformity	All	N.S.
3.308. Physical and chemical stability	Furnace clinker	B.S. 1165
	Foamed slag	B.S. 877
	Others	N.S.
3.309. Behaviour at high temperatures	All	N.S.
3.310. Resistance to weathering	All	N.S.

[1] N.S.—no British Standard.
[2] B.S. 877 : 1939, " Foamed Blast-furnace Slag for Concrete Aggregate."
[3] B.S. 1165 : 1947, " Clinker Aggregate for Plain and Precast Concrete."

of the requisite composition. Secondly, where comparisons are required between different aggregates, the behaviour of comparable mixes under the same test conditions may give the required information far more conveniently than any other way.

Standard methods of conducting tests on given specimens exist for the following significant properties, but there is no accepted standard method of making test specimens of lightweight concrete.

3.51. Unit weight
3.52. Strength
3.53. Drying shrinkage and moisture movement
3.55. Thermal conductivity

There are no accepted British Standard tests for :—

3.54. Moisture absorption and retention
3.56. Behaviour at high temperatures
3.57. Resistance to weathering

3.6. *Tests on lightweight aggregates and concrete*

There are at present no suitable British Standard tests to determine some of the properties which may be significant.

This report includes a number of recommendations for the development of existing tests and the formulation of new tests, where such a course appears desirable.

The procedures for carrying out some of the tests listed above are summarized in Appendix D.

3.7. *Discussion of tests on lightweight aggregates*

3.701. *Unit weight*

The test for unit weight as performed for gravel or stone aggregates (B.S. 812, clause 19) cannot be used for lightweight aggregates. It is likely that the test included in B.S. 877, Appendix A, for foamed slag (D–1.1) could be applied to other types of lightweight aggregate. It is necessary for this to be verified or other suitable tests devised, and the Committee recommends that research be undertaken. (See also recommendation 4.201.)

3.702. *Grading*

With the stronger aggregates (clinker, expanded clays, and normal foamed slags) sieving methods may be used for the determination of particle size distribution. With the weaker and more friable aggregates, these methods may cause breakage of the particles. The Committee does not know how far sieving methods may be applicable, and a suitable test is required for those aggregates for which they are not applicable. The Committee recommends that research be undertaken. (See also recommendation 4.202.)

3.703. *Particle shape and surface characteristics*

Many lightweight aggregates consist of angular particles with pitted or cavernous surfaces due to the material being reduced to size by crushing. Certain types are specially treated in production to give particles which are rounded with relatively smooth surface skins. These properties have a marked influence on the operation of mixing, and on the workability of the resulting concrete. There is no agreed nomenclature for describing particle shape and surface characteristics in the case of lightweight aggregates, and the Committee recommends that consideration should be given to standardizing a suitable terminology. (See also recommendation 4.203.)

3.704. *Water absorption*

All lightweight aggregates are cellular or porous, but various types of pores exist. Some pores are closed and inaccessible to water, others are open and may be filled with water under certain conditions. The proportion of accessible pores and the rate at which these fill or give up water affects aggregate preparation, concrete mixing, curing, and the properties of the concrete. There are no standard tests for water absorption of

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lightweight aggregates. The tests laid down in B.S. 812 are not applicable and the Committee recommends that research be pursued. (See also recommendation 4.204.)

3.705. *Strength and compressibility*

These properties of an aggregate vitally affect the properties of any concrete made with the aggregate. No satisfactory tests exist for assessing these properties by direct testing of the aggregate, and it is advisable to test the particular mix of concrete to be used for those properties which are affected by the strength and compressibility of the aggregate ; namely the strength, drying shrinkage and moisture movement, behaviour at high temperatures, and resistance to weathering.

3.706. *Impurities*

Tests for the impurities which are peculiar to clinker and foamed slag have been developed and included in the British Standards for those materials and the Committee considers these tests to be acceptable (D–2). It is possible for soluble salts, combustible matter, heavy material, silt or clay, to be present in any lightweight aggregate and tests for these impurities applicable to lightweight aggregate are required.

Furnace clinker and expanded clays and shales may contain pellets of quicklime which may be troublesome by causing " popping " or scaling at the surface of concrete made with that aggregate by delayed expansion. Particles of iron or iron-containing compounds may similarly cause " popping " by oxidation, but they are chiefly troublesome by causing staining. No standard tests exist for these impurities and the Committee recommends that research be undertaken. (See also recommendation 4.205.)

3.707. *Uniformity*

There is no test available for the assessment of the uniformity of aggregates in bulk, and the Committee recommends that research be undertaken. (See also recommendation 4.206.)

3.708. *Physical and chemical stability*

The existing tests for stability (D–3) are directed to the problems peculiar to clinker and foamed slag and are not applicable to other aggregates. So far as the Committee is aware, there is no indication from practical experience that any other lightweight aggregates give rise to troubles on account of chemical or physical instability.

3.709. *Behaviour at high temperatures*

See Item 2.608. (See also recommendation 4.207.)

3.710. *Resistance to weathering*

The resistance to weathering of a lightweight aggregate may be of some interest where aggregate is to be stored in the open, but no standard tests exist for this property. It does not appear likely that tests of the

resistance of the aggregate will necessarily be of much help in assessing the resistance to weathering of concrete made with the aggregate.

3.8. *Discussion of tests on lightweight concrete*

Lightweight concretes are used for building blocks and other machine-moulded products, and are also cast in situ. The mixes used for the two classes of work are very different, one being dry and the other plastic, and the behaviour of an aggregate in a mix of one type is not necessarily the same as in the other. The method of mixing is dependent upon the particular aggregate, the mix proportions, and workability of the mix. The forming of concrete blocks in a machine is by pressure, tamping or vibration, or combinations of these. For in-situ work the methods of compaction applicable to normal concrete are used. The proportions of voids accordingly differ and, therefore, if test specimens made in the laboratory are to bear any relation to the concrete produced on the full scale, comparable methods of concrete making and compaction must be used.

If the laboratory concrete is intended for the purpose of testing the aggregate, then a mix, method of mixing, and method of compaction chosen arbitrarily, may suffice for comparative purposes.

There are no British Standards for the mixing and making of lightweight-concrete specimens either for the testing of concrete or the comparison of aggregates, and the Committee recommends that research be pursued. (See also recommendation 4.208.)

3.81. *Unit weight, strength, and drying shrinkage and moisture movement*

Whilst there are no British Standard methods of performing these tests which are specific to lightweight concrete, the methods of test given for concrete blocks in B.S. 2028* are applicable, the test specimen being treated as a block for the purpose of the test.

3.82. *Water absorption and retention*

There are no standard methods of test and the Committee recommends that research be pursued. (See also recommendation 4.209.)

3.83. *Thermal conductivity*

The thermal conductivity of lightweight concrete in the condition of humidity normal in structures merits far greater attention than it has hitherto been given, and standard methods of test for its determination are required. The difficulty of drying out concrete structures is well known, and with the more water-retentive aggregates and concretes, the thermal conductivity in situ may be considerably greater than that of a dry specimen. The Committee recommends that research be pursued in order to provide data covering the range of humidity conditions likely to occur in practice. (See also recommendation 4.210.)

* B.S. 2028 : 1953, "Precast Concrete Blocks."

3.84. *Behaviour at high temperatures*
See Item 3.709.

3.85. *Resistance to weathering*
No standard tests are available. The Committee is aware of the difficulties involved but feels that the matter is so important that it is hoped research will be pursued. (See also recommendation 4.211.)

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*15. T. Whitaker
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*16. T. Whitaker
Lightweight concretes : a review.
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17. Cement and Concrete Association, Library Record Ch. 20 (3/51), Bibliography on Lightweight Concrete.

Part 4. Recommendations

4.1. *For aggregates other than lightweight*

4.101. That research be carried out to develop a method of test that will give a quantitative value for the surface texture of aggregates.

4.102. That the test for roundness or angularity of aggregates described in Appendix A (A–1.3) be used when it is required to measure this property of aggregates, and that work on these lines to include aggregates larger than $\frac{3}{4}$ in. should be pursued.

4.103. That the procedure for the test for the specific gravity and absorption of coarse aggregate (B.S. 812 : 1951, clause 16) be modified in the light of recent published research (see footnote to Section 2.202).

4.104. That research is required to formulate a test for the uniformity of aggregates in bulk by determining the variation of the mechanical properties of aggregates throughout the bulk.

4.105. That research is required to determine what impurities in aggregates are harmful and if present, up to what limits they are permissible, and to devise suitable tests.

* These papers contain extensive bibliographies.

4.106. That research is urgently required with a view to simplifying fire resistance testing of aggregates and concrete so that their properties may be correlated with the behaviour of structural elements made of these materials.

4.107. That where extensive new work is to be undertaken using aggregates of which there is no previous experience, tests should be carried out to determine if reactive silica is present. This is particularly important if it is intended to use a high-alkali cement.

4.108. That further research be undertaken to develop a reliable and rapid method of testing for the presence of reactive silica liable to cause expansion.

4.109. That research should be pursued, in spite of the difficulties involved, to formulate a test for determining the resistance of concrete to weathering.

4.110. That research should be undertaken to formulate a test for determining the resistance of concrete to wear. The test for paving flags described in B.S. 368 should be amended so that the accumulated dust is removed continuously during the course of the test.

4.2. *For lightweight aggregates*

4.201. That research be undertaken to verify whether the test for unit weight for foamed blast-furnace slag (D—1.1) can be applied to other aggregates or to devise other suitable tests.

4.202. That research be undertaken to formulate a suitable method of grading those lightweight aggregates which are too weak or friable to permit the use of sieving methods.

4.203. That consideration be given to standardizing a suitable terminology for describing particle shape and surface characteristics of lightweight aggregates.

4.204. That research to formulate tests for water absorption of lightweight aggregates be pursued.

4.205. That research be undertaken to devise standard tests for the assessment of impurities in lightweight aggregates other than clinker and foamed slag, and for detecting the presence of pellets of quicklime or particles of iron or iron-bearing compounds which may be deleterious in any lightweight aggregate.

4.206. That research be undertaken to formulate a test to determine the uniformity of lightweight aggregates in bulk.

4.207. That research is urgently required with a view to simplifying fire resistance testing of lightweight aggregates and concrete so that their properties may be correlated with the behaviour of structural cements made of these materials.

4.208. That research into methods of mixing and making lightweight concrete specimens for testing concrete or comparing aggregates be pursued.

4.209. That research on testing for water absorption and retention of lightweight concrete be pursued.

4.210. That research into the thermal conductivity of lightweight concrete be pursued in order to provide data covering the range of humidity conditions likely to occur in practice.

4.211. See recommendation 4.109.

APPENDIX A: SUMMARY OF TESTING PROCEDURES FOR AGGREGATES OTHER THAN LIGHTWEIGHT

The details given below are insufficient as working instructions for which reference should be made to the standard procedures quoted.

A–1. *Particle shape and surface texture*

Reference: B.S. 812 : 1951, "Sampling and Testing of Mineral Aggregates, Sands and Fillers," clause 10.

A–1.1. *Description*

The particle shape can be described verbally by one of the following standard classifications:

rounded;
irregular;
angular;
flaky.

Surface texture may similarly be described by the following standard group classifications:

glassy;	crystalline;
smooth;	rough;
granular;	honeycombed and porous.

A–1.2. *Measurement of aggregate shape (flakiness and elongation indices)*

Reference: B.S. 812 : 1951, "Sampling and Testing of Mineral Aggregates, Sands and Fillers," clause 15.

The particle shape of aggregates is determined by the percentage of flaky and elongated particles that they contain. These are defined respectively as particles whose least dimension is less than 0·6 of their mean size and whose greatest dimension is more than 1·8 times their mean size.

The aggregate is first sorted on square-aperture British Standard test sieves* into a number of closely limited particle-size groups and the samples

* B.S. 410 : 1943, "Test Sieves."

in each group are tested for length and thickness on appropriate gauges. The flaky particles can alternatively be separated on slotted sieves of the appropriate dimensions.

The flakiness index is the total weight of material passing the thickness gauges or sieves, expressed as a percentage of the total weight of the sample.

The elongation index is the total weight of material retained on the length gauges expressed as a percentage of the total weight of the sample.

A–1.3. *Roundness or angularity*

Reference : Shergold, F. A., " The Percentage Voids in Compacted Gravel as a measure of its Angularity."

Mag. Conc. Res. 1953, 5 (13), 3–10.

A metal cylinder of about $\frac{1}{10}$ cu. ft capacity is filled with aggregate in a specified manner and the percentage voids are determined from the following formula :

$$V = 100\left(1 - \frac{W}{C \times P}\right)$$

where V denotes percentage of voids,

W denotes mean weight of aggregate in the cylinder (g),

C denotes weight of water required to fill the cylinder (g),

P denotes " apparent " specific gravity of the aggregate on dry basis (B.S. 812 : 1951, clauses 16, 17).

Then :—

$$\text{Angularity number} = V - 33$$

This procedure is at present limited to single-sized aggregates within the range $\frac{3}{16}$ in. to $\frac{3}{4}$ in.

A–2. *Water absorption, specific gravity, and unit weight*

Reference : B.S. 812 : 1951, " Sampling and Testing of Mineral Aggregates, Sands and Fillers," Part Four.

Tests are made on 7-lb. samples of crushed aggregate or gravel by drying at 100° to 110°C for 24 hours and then immersing in water for 24 hours.

The specific gravity is calculated from the formula $A/(A - C)$ where A is the dry weight of the aggregate and C is the apparent weight in water.

The water absorption is a percentage and is calculated from the formula $\left(\frac{B - A}{A}\right) \times 100$ where A is the dry weight and B is the saturated weight of the sample.

In the case of aggregate finer than $\frac{3}{8}$ in. British Standard test sieve or sand, a sample of the aggregate is weighed (a) as received, (b) after drying for 24 hours at 100° to 110°C, (c) after soaking in water for 24 hours, and (d) immersed in water in a pycnometer of known capacity. From these figures, the moisture content, water absorption, and specific gravity are

calculated on either a dry or a saturated basis according to the purpose for which the results are required.

The unit weight of aggregate in lb/cu. ft is determined from the weight of compacted aggregate contained in a standard measure of $\frac{1}{10}$, $\frac{1}{2}$, or 1 cu. ft capacity, depending on the size of aggregate.

A–3. *Grading*

Reference : B.S. 812 : 1951, "Sampling and Testing of Mineral Aggregates, Sands and Fillers," clause 11.

The grading of aggregate is determined by shaking it for not less than 2 min on each of such square-aperture British Standard test sieves* as are appropriate to define the aggregate size. Perforated-plate sieves are used for all sizes from 3 in. to $\frac{3}{16}$ in. and woven-wire sieves for smaller meshes. The results are calculated and reported as :—

(i) The percentage by weight of the total sample passing one sieve and retained on the next smaller sieve, to the nearest 0·1%.

(ii) The cumulative percentage by weight of the total sample passing each of the sieves to the nearest whole number. It is recommended that cumulative percentage figures should be used for comparison with specification requirements, or for reporting results graphically.

A–4. *Petrological characteristics*

Reference : B.S. 812 : 1951, "Sampling and Testing of Mineral Aggregates, Sands and Fillers," clause 9.

The following Group classification is adopted for the convenience of producers and users of aggregates :—

Names of Groups

Artificial	Hornfels
Basalt	Limestone
Flint	Porphyry
Gabbro	Quartzite
Granite	Schist
Gritstone	

The correct identification of a rock and its placing under the appropriate Group should be left to the decision of H.M. Geological Survey and Museum or a qualified geologist.

A–5. *Mechanical strength*

A–5.1. *Resistance to impact*

Reference : B.S. 812 : 1951, "Sampling and Testing of Mineral Aggregates, Sands and Fillers," clause 23.

The apparatus consists of a $3\frac{15}{16}$-in.-dia. hammer weighing 30 lb. falling

* B.S. 410 : 1943, "Test Sieves."

freely between guides a distance of 15 in. on to a sample of the aggregate held in a cylindrical cup, 4 in. in diameter and 2 in. deep.

The test is carried out by subjecting about ¾ lb. of ½-in. aggregate to 15 blows of the hammer. The result is expressed as the percentage of fines formed (passing No. 7 British Standard test sieve) to the total sample, either by weight or by volume.

A–5.2. *Resistance to crushing*

Reference: B.S. 812 : 1951 "Sampling and Testing of Mineral Aggregates, Sands and Fillers," clause 25.

About 7 lb. of ½-in. aggregate is dried at 100°C–110°C and placed in a 6-in.-dia. hardened steel cylinder with a closely fitting ram or plunger. A load of 40 tons is then applied to the aggregate by means of a compression testing machine.

The test result is expressed in terms of the percentage of fines formed (passing No. 7 British Standard test sieve) to the total sample weight.

Other nominal sizes of aggregate can be tested if ½-in. aggregate is not available, but the load, diameter of the cylinder, and the sieve for separating the fines are then varied.

A–6. *Impurities*

Reference: B.S. 812 : 1951, "Sampling and Testing of Mineral Aggregates, Sands and Fillers," clauses 12, 13, 14, and 22.

A–6.1. *Clay, fine silt, and fine dust*

The tests for the determination of clay, fine silt, and fine dust in aggregates as specified in B.S. 812 : 1951 are described in clauses 12, 13, and 14.

The methods described in clauses 12 and 13 are laboratory methods.

Clause 12 describes the Sedimentation Method. It depends on sedimentation and pipette sampling.

Clause 13 describes a method which determines the amount of material which passes a No. 200 B.S. sieve.

To obtain a fairly rapid indication of the proportion of these impurities in an aggregate, the Field Settling Test described in clause 14 may be used. In this test about 50 ml. of a solution of common salt in water (about one teaspoonful of salt to one pint of water, giving approximately a 1% solution) is placed in a 200 ml. B.S. measuring cylinder. Fine aggregate as received is then added until the volume of aggregate is 100 ml. The total volume is then made up to 150 ml. by the addition of more of the salt solution. The mixture is then shaken vigorously to disperse the adherent clayey particles and afterwards allowed to settle for 3 hours. The clay, silt, and dust settle on top of the sand and the height of this layer can be expressed as a percentage of the height of the sand below. This approximate test is not generally applicable to crushed-stone sands.

A–6.2. *Organic materials*

An approximate method of indicating the amount of organic impurities in sand is given in B.S. 812 : 1951, clause 22. It is carried out by observing the discolouration produced by the sample on a standard solution of sodium hydroxide. A graduated standard five-colour chart is given in B.S. 812, and any sand producing a colour darker than No. 3 on this scale is regarded as suspect.

A–7. *Chemical stability : tests for alkali-reactive aggregate*

A–7.1. *Rapid chemical test*

References : R. C. Mielenz, K. T. Greene, and E. J. Benton, " Chemical Test for Reactivity of Aggregates with Cement Alkalies ; Chemical Processes in Cement-Aggregate Reaction." J. Amer. Conc. Inst. 1947, 19 (Proc 44), 193–224.

A.S.T.M. " Tentative Method of Test for Potential Reactivity of Aggregates (Chemical Method) "—C289–52T.

A sample of aggregate is crushed to 52–100 mesh and immersed in a normal solution of NaOH (25g aggregate to 25ml. NaOH) at 80°C, for 24 hours. The solution is then filtered off, analysed for dissolved silica, and the reduction in alkalinity (i.e., the reduction in strength of the alkali solution) also determined by titration. The ratio of dissolved silica to reduction in alkalinity is calculated. If less than unity the aggregate is harmless ; if it exceeds unity the aggregate is considered to be reactive. It is important, however, to note that it does not follow that it will be *expansively* reactive with cement and confirmation by the expansion bar test is necessary.

A–7.2. *Expansion bar test*

References : F. E. Jones and R. D. Tarleton, " Reactions between Aggregates and Cement. Part III. Alkali-Aggregate Interaction : The Expansion Bar Test." National Bldg. Studies Res. Paper No. 17, H.M.S.O., 1952.

A.S.T.M., " Tentative Method of Test for Potential Alkali Reactivity of Cement-Aggregate Combinations "—C.227–52T.

Mortar bars measuring 1 in. by 1 in. by 10 in. are prepared using a cement/aggregate ratio of 1 : 2 by weight, and a high alkali-content in the mix. They are stored at 20°C in sealed containers, over but not in contact with water. Initial lengths are determined immediately after removal from the moulds at 24 hours. Subsequently measurements are made at 1 week and at 2, 4, 6, 9, and 12 months. If the expansion exceeds 0·05% in 6 months or 0·1% in 12 months, the aggregate is to be classed as expansively reactive.

Appendix B: summary of testing procedures for concrete

The details given below are insufficient as working instructions for which reference should be made to the standard procedures quoted.

B–1. *Strength*

Reference: B.S. 1881 : 1952, "Methods of Testing Concrete," Part 8.

A 6-in. cube of concrete made and cured in a standard manner is placed in a testing machine and loaded at a rate of approximately 2,000 lb/sq. in./min until failure.

The compressive strength of the cube is calculated by dividing the maximum load carried during the test by its cross-sectional area. The result is expressed in lb/sq. in. to the nearest 50 lb/sq. in.

B–2. *Unit weight*

Reference: B.S. 1881 : 1952, "Methods of Testing Concrete," Part 5.

A sample of freshly mixed concrete is compacted in a cylinder with a capacity of ½ cu. ft for aggregates up to 1½ in. and of 1 cu. ft for larger aggregates. The concrete is compacted by using a 4 lb. tamping bar.

The unit weight of the cylinder is determined by dividing the weight of concrete in the cylinder by the capacity of the cylinder, the result being expressed in lb/cu. ft.

B–3. *Thermal conductivity*

References: E. Griffiths, "The Measurement of the Therma Conductivity of Materials Used in Building Construction." J Instn Heating and Ventilating Engrs, vol. 10 (1942), pp. 106–108 B.S. 874 : 1939, "Definitions of Heat Insulating Terms and Method of Determining Thermal Conductivity and Solar Reflectivity."

Two specimens of the concrete, 8-in. square, are placed on either sid of a hot plate, with two cold plates in contact with the other faces of th specimens. The complete unit includes a refrigerating plant, thermostati devices, and a centrifugal circulating pump. The heat flow is measure and the thermal conductivity is reported as g-cal/sq. cm/sec for 1 cn thickness and 1°C difference in temperature.

B–4. *Drying shrinkage and moisture movement*

Reference: B.S. 1881 : 1952, "Methods of Testing Concrete, Part 16.

Specimens of concrete between 6 in. and 12 in. long and about 3 in square in cross-section are used. The following variations in length of th specimens are reported :—

(*a*) Initial drying shrinkage: the difference between the lengths when moulded and cured under specified conditions and the length when oven dry.

(*b*) Drying shrinkage: the difference between the length when the specimen is cut from matured concrete and the length when oven dry.

(*c*) Moisture movement: the difference between the length when oven dry and the length when subsequently saturated.

B–5. *Resistance to wear*

Reference: B.S. 368 : 1936, "Precast Concrete Flags."

The rate of wear of precast concrete flags is tested by fixing samples 2 ft by 1 ft across openings 22 in. by 10 in. in the sides of a rectangular container with solid ends.

One thousand hard steel or chilled cast-iron balls, ½-in. dia., are placed in the container which is then rotated on a central shaft at a regular speed of 60 r.p.m. for 24 hours. After removing the dust formed the container is rotated for a further 24 hours in the reverse direction.

When tested in the manner described, the wear on the face of a sample shall be uniform and shall not result in a total loss in weight of more than 2 lb.

Appendix C: summary of sampling procedure

The details given below are insufficient as working instructions for which reference should be made to the standard procedures quoted.

C–1. *Sampling of aggregates*

Reference: B.S. 812: "Sampling and Testing of Mineral Aggregates, Sands and Fillers," Part One.

The essential points of the instruction are that samples should preferably be taken at the time of loading or unloading and that small samples should be taken at regular intervals and mixed to make a large composite sample, which is finally reduced to the required size by quartering or by means of a sample-splitter.

If samples must be taken from a bin or stock-pile, the small samples should be taken from places evenly distributed over the pile, removing about 1 ft of material from the surface before taking the sample, and avoiding any patches of segregated material.

The operation known as "quartering" is the formation of a roughly conical heap from the sample, dividing it into four quadrants, and taking two diagonally opposite quadrants to form the reduced sample, which is further reduced by the same method until of the required size.

C–2. *Sampling of concrete*

Reference: B.S. 1881 : 1952, "Methods of Testing Concrete," Part 1.

Samples can be taken from the stream of concrete being discharged from a mixer or from concrete already deposited.

In the first case the sample is made up of at least three increments taken from a single discharge and in the second case from not less than five increments taken from different points in the batch.

In both cases the increments are mixed to form composite samples in a way which will ensure uniformity.

Appendix D: summary of testing procedures for lightweight aggregate

The details given below are insufficient as working instructions for which reference should be made to the standard procedures quoted.

D–1. *Unit weight*

D–1.1. *Foamed blast-furnace slag*

Reference: B.S. 877 : 1939, "Foamed Blast-furnace Slag for Concrete Aggregate," Appendix A.

A rigid box, rectangular in shape, preferably of wood, of interior volume 1 cu. ft is used. The sample for test is to be room dry and to pass a $\frac{1}{2}$-in.-mesh B.S. sieve and be retained on a $\frac{3}{16}$-in.-mesh B.S. sieve.

The sample is filled into the box by means of a shovel or scoop, the material being deposited lightly without tamping or shaking the box or knocking the sides. The filling is continued until the box is filled to overflowing and the surplus struck off with a straight edge. The net weight of the aggregate in the box is taken as the weight per cu. ft.

D–2. *Impurities*

D–2.1. *Sulphate*

D–2.11. *Sulphate in clinker aggregate*

Reference: B.S. 1165 : 1947, "Clinker Aggregate for Plain and Precast Concrete," Appendix B.

A sample is ground to pass 100-mesh B.S. sieve and dried to constant weight at 100–110°C. One g of this material is boiled with hydrochloric acid, filtered and the sulphur-trioxide content of the filtrate determined and expressed as a percentage of the original sample.

D–2.12. *Sulphate in Foamed Blast-Furnace Slag*

Reference: B.S. 877 : 1939, "Foamed Blast-furnace Slag for Concrete Aggregate," Appendix E.

A number of pieces weighing a total of 20 g are taken from a sample passing $\frac{1}{2}$-in.-mesh B.S. sieve and retained on $\frac{3}{8}$-in. B.S. sieve, and placed in a 250 ml. flask in 230 ml. of de-aerated distilled water and the flask tightly stoppered. The flask is stored for 24 hours, being shaken frequently during the last 4 hours. The contents are then filtered, the

filtrate acidified with hydrochloric acid and boiled. The sulphur-trioxide content of the filtrate is determined and expressed as a percentage of the original sample.

D–2.2. *Combustible material*

D–2.21. *Loss on ignition of clinker aggregate*

Reference: B.S. 1165 : 1947, "Clinker Aggregate for Plain and Precast Concrete," Appendix C.

A 5 lb. sample of the clinker is first ground to pass 18-mesh B.S. sieve, and of this ½–1 lb. further ground to pass 72-mesh B.S. sieve. One gramme of this material is first dried at 100–110°C to constant weight. It is then ignited first at 700–800°C for 4 hours and then at 950–1,050°C for a further hour, then allowed to cool in a desiccator and weighed.

The loss of weight on ignition is expressed as a percentage of the dried weight.

D–2.22. *Determination of coke in foamed blast-furnace slag*

Reference: B.S. 877 : 1939, "Foamed Blast-furnace Slag for Concrete Aggregate," Appendix D.

A sample is crushed to pass 100-mesh B.S. sieve. Approximately 100 g of this material is heated to 400°C for 2 hours and weighed. It is then heated at 600–650°C for a further 5 hours and again weighed. The loss in weight over this 5-hour period expressed as a percentage of the weight at the beginning of the period is taken as the percentage of coke.

D–2.3. *Heavy impurities*

D–2.31. *Heavy impurities in foamed blast-furnace slag*

Reference: B.S. 877 : 1939, "Foamed Blast-furnace Slag for Concrete Aggregate," Appendix B.

The method depends upon the elutriation of a 100-g sample of dried aggregate of size passing ½-in.-mesh B.S. sieve and retained on ⅜-in.-mesh B.S. sieve in a tube of specified dimensions in an upward stream of water of specified rate of flow. The operation is performed in a prescribed manner. The residue after elutriation is dried and weighed and expressed as a percentage of the original weight of the sample.

D–3. *Physical and chemical stability*

D–3.1. *Soundness of clinker aggregate*

Reference: B.S. 1165 : 1947, "Clinker Aggregate for Plain and Precast Concrete," Appendix D.

The method involves making pats of a mixture of Portland cement, plaster of Paris and clinker, the clinker sample being ground to pass 72-mesh B.S. sieve. The method of gauging and the proportions are specified. The pats are formed on glass plates, and are kept when set in a moist atmosphere. Unsoundness is indicated by the cracking or lifting of the edges of the pats within a period of 7 days. For testing class A and B clinker,

one part of cement-plaster mixture is mixed with five parts by volume of clinker, whilst for class C clinker the proportions to be used are one to three parts by volume.

D–3.2. *Stability of foamed blast-furnace slag*

Reference: B.S. 877 : 1939, "Foamed Blast-furnace Slag for Concrete Aggregate," Appendix C.

The test is performed on the heavy impurities forming the residue of the elutriation test together with pieces taken from the original sample to make a total of twenty pieces in all.

The pieces are dried and weighed and then placed to stand in a beaker in distilled water for 14 days. At the end of storage the contents of the beaker are decanted through a $\frac{1}{8}$-in.-mesh B.S. sieve. The material retained on the sieve is dried and the loss in weight expressed as a percentage of the original weight.

ELECTION OF ASSOCIATE MEMBERS

The Council at their meetings on the 15th February, 1955, and the 15th March, 1955, in accordance with By-law 14, declared that the undermentioned had been duly elected as Associate Members.

Adye, Alan Michael, M.A. (*Cantab*), Grad.I.C.E.
Amesbury, Kenneth, B.Sc. (*Bristol*), Grad.I.C.E.
Ballantine Dykes, Joseph.
Beard, Peter Willmot, B.Sc.(Eng.) (*London*), Grad.I.C.E.
Briggs, John Kenneth, B.Sc.Tech. (*Manchester*).
Broadley, Robert, Grad.I.C.E.
Broome, Michael Rowland, B.Sc. (Eng.) (*London*).
Cameron, Angus Murray, Grad.I.C.E.
Cameron, John Charles Finlay, Grad.I.C.E.
Carr, John Hallett, Grad.I.C.E.
Cassidy, Kenneth Evan.
Clamp, Eric John Sydney, Stud.I.C.E.
Cowbourne, Dennis Stuart, B.Sc. (Eng.) (*London*), Grad.I.C.E.
Crosthwaite, Donald Rothery, B.A. (*Cantab*), Grad.I.C.E.
De Silva, Muttuwa Sarukalige Maitripala, B.Sc.(Eng.) (*London*).
Dixon, Alan James Burney, B.Sc. (*Manchester*), Grad.I.C.E.
Duquemin, David John, Stud.I.C.E.
Edwards, David William Wood, B.Sc.(Eng.) (*London*), Grad.I.C.E.
Esler, Robert Acheson, B.Sc. (*Belfast*), Grad.I.C.E.
Fairley, Archibald Thompson, B.Sc. (*Edinburgh*).
Ganly, Paul, B.A. (*Cantab*).
Goodman, David Farrand, B.Eng. (*Liverpool*), Grad.I.C.E.
Gorringe, Peter Brian.
Gray, Charles Alexander Menzies, M.E. (*Sydney*).
Hamley, Elwyn William, B.Sc.(Eng.) (*London*), Grad.I.C.E.
Heeley, John Mark, B.Sc.(Eng.) (*London*), Grad.I.C.E.
Henderson, Ronald Vernon, B.Sc. (*Belfast*), Grad.I.C.E.
Hubbard, Harold William Charles, B.Sc.Tech. (*Manchester*), Grad.I.C.E.
Jones, Edward Malcolm, B.Sc. (*Wales*), Grad.I.C.E.
Jones, Thomas Aneurin Rhys, Grad. I.C.E.
Kinsella, James, M.Sc. (*Belfast*), B.E. (*National*), Grad.I.C.E.
Lawrence, John Englefield, Grad. I.C.E.
McCallum, Archibald Hugh, B.Sc. (*Glasgow*), Grad.I.C.E.
McKerracher, Ian, Grad.I.C.E.
McKinlay, David Gemmell, B.Sc. (*Glasgow*), Grad.I.C.E.
McLeish, William Purves, B.Sc. (*Edinburgh*), Grad.I.C.E.
Martin, John Ellery, Grad.I.C.E.
Mitchell, William Humphry Bevan, B.A., B.A.I. (*Dublin*).
Mobbs, John Cedric, B.Sc.(Eng.) (*London*), Grad.I.C.E.
Morris, David Austin, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Muir, Ian Stanley, B.Sc. (*Durham*), Grad.I.C.E.
Oates, Frank Laurence, Stud.I.C.E.
Ollerhead, Geoffrey Charles, B.Sc. (*Birmingham*).
Payne, Norman John, M.B.E., B.Sc. (Eng.) (*London*).
Power, John Charles, M.A. (*Cantab*), Grad.I.C.E.
Pratt, Geoffrey Cyril, B.Sc.(Eng.) (*London*), Grad.I.C.E.
Pritchard, Gerald Caradoc Elmo, B.Sc. (*Wales*).
Shapcott, Phillip Charles.
Shores, Kenwyn Douglas Tatham, B.E. (*New Zealand*), Grad.I.C.E.
Smith, Harold Geoffrey, Grad.I.C.E.
Sparks, John Albert Whitaker, M.C., B.Sc. (*London*).
Swan, Ronald, B.Eng. (*Sheffield*), Grad.I.C.E.
Thompson, Leslie Henry, Grad.I.C.E.
Walker, Hubert Eaton, M.Eng. (*Liverpool*), Grad.I.C.E.
Waterhouse, Arnold, B.Sc.(Eng.) (*London*).
Weller, Rodney Robert, B.Sc.(Eng.) (*London*), Stud.I.C.E.
White, Kenneth Francis, B.Sc.(Eng.) (*London*), Grad.I.C.E.
Wilson, Thomas Gilmour, B.Sc. (*Belfast*), Grad.I.C.E.
Young, Thomas Leslie, B.A., B.A.I. (*Dublin*), Grad.I.C.E.

DEATHS

It is with deep regret that intimation of the following deaths has been received.

Members

Robert Glover Baxter, O.B.E., M.A. (E. 1927, T. 1946).
Sir Ernest Albert Seymour Bell, C.I.E., F.C.H. (E. 1895, T. 1913) (former Member of Council).
Ernest Alexander Black (E. 1915, T. 1941).
Hubert Cecil Booth (E. 1897, T. 1910).
Robert Bruce (E. 1904, T. 1926).
Herbert Chatley, D.Sc.(Eng.) (E. 1920, T. 1928).
Walter Ponsonby Shaw Cockle, B.Sc.(Eng.) (E. 1928, T. 1945).
Sir Maurice Edward Denny, Bart., K.B.E., B.Sc. (E. 1918, T. 1924).
John Samuel Duncan, B.Sc. (E. 1936, T. 1945).
John Thomas Dungate Fillingham (E. 1911, T. 1931).
James M'Farquhar Petters (E. 1900, T. 1910).
Frederick Gerald Rappoport (E. 1914, T. 1925).
Leopold Penrose Ridgway (E. 1912, T. 1927).
John Charles Telford, O.B.E. (E. 1910, T. 1922).
John Terrace (E. 1924).

Associate Members

Soden William Biden (E. 1908).
Leonard Onslow Cope (E. 1925).
Ronald Murray Drynan (E. 1925).
Percy Richard Duncan, M.A., B.Sc. (E. 1938).
William Ainslie Foulis (E. 1901).
Frederick William Jones (E. 1907).
Robert Owen Jones (E. 1912).
Jan Kacirek, B.Sc.(Eng.) (E. 1954).
Alexander Hamilton Knox (E. 1921).
Albert Edward Charles Leppard, B.Sc.(Eng.) (E. 1922).
Leonard Watson Pye (E. 1915).
Harvey Robert Sayer (E. 1917).
Walter McNeil Shimmin (E. 1920).
Stanley White, B.Sc. (E. 1940).
William Percy Wordsworth, B.Sc. (E. 1917).

Graduates

Edward Shepherd Darling, B.A. (A. 1952).

Students

John Michael Harrison (A. 1952).
John Gavin Johnston (A. 1947).

CORRESPONDENCE

on a Paper published in Proceedings, Part I, May 1954

Paper No. 6001

"Concrete Quality Control at Woodhead New Tunnel" †

by

Frederick Andrew Sharman, B.Sc.(Eng.), A.M.I.C.E.

Correspondence

Mr W. J. Sivewright observed that the considerable amount of investigation carried out, and the volume of interesting results thereby achieved were of the nature of an academic exercise in relation to what was presumably the prime object—namely, to ensure as economically as possible that the strength of the concrete used in the work did not fall below the minimum required by the structural design.

The criticism was valid for many other works besides that which formed the subject of the Paper.

Concrete differed from other materials of construction in that the specific strength of the component parts was not necessarily a measure of the strength of the whole; in fact the material had not come into being until the structure was complete and ready for load-bearing.

Generally, testing the finished concrete was impracticable and in most classes of structure the correction of faulty sections was virtually impossible as the Author himself had stated on p. 570.

Sometimes, as in the case of the Woodhead Tunnel, the effect of an occasional bad batch was mitigated by the mixing of successive batches in the structure. In other cases, as in a framed structure, one faulty batch could lead to a dangerous loss of strength at a vital point.

On p. 570 the Author had stated that the maximum allowable proportion of batches below 2,750 lb/sq. in. was limited to 8% and that special action was to have been called for if any result fell below 2,300 lb/sq. in. What action had in fact been contemplated, in view of the 28-day testing period? On p. 571 it was stated that the specified 8% below 2,750 lb/sq. in. had not been appreciably exceeded. Had any action been taken when it was exceeded? Had any consideration been given to accelerated tests,

† Proc. Instn Civ. Engrs, Part I, vol. 3, p. 564 (Sept. 1954).

e.g., by steam-curing the cubes, so as to lessen the period which had to elapse before action could be taken in respect of low-strength concrete?

It was a truism that the failures of one job provided the lessons for the success of the next, and one might conclude that the test results for concrete on one job would be of more value as the basis of the mix design for the next job, than as control on the job during which they were obtained; except that that would be true only if the cement, aggregates, conditions of placing, etc., were identical in the two cases.

Nevertheless it was very desirable to know the properties of the finished concrete before concreting commenced.

On large projects could not that be achieved by a large-scale series of tests before the work began, simulating the actual conditions of placing so far as possible? The concrete produced for those tests would not be wholly wasted, for on a job of any considerable magnitude the preliminary site work required a considerable production of concrete for hut floors, plant bases, drain surrounds, etc., where concrete quality was of little importance. Such a procedure would add somewhat to the cost of the work but that was to be regarded as the price to be paid for foreknowledge.

In that connexion it was well worth bearing in mind the Author's statement that 2 lb. of cement per cu. yd involved £1,200 on a contract worth £4½ million. How did that compare with the cost of the tests carried out?

In the smaller job the mix to be used would have to be assessed from a knowledge of the available aggregates and cement and from past experience of concrete made with them, the safety (or "ignorance") factor of design being adjusted accordingly. That was, of course, precisely what was done on the majority of works, where quality control was not practised.

Mr E. R. Giles disagreed with the Author's interpretation of the asymmetry of the Histogram (Fig. 5). The Author's explanation rested on his conclusion that batching errors, notably in excess cement, gave a high number of samples of low water/cement ratio, and also on the fact that sub-standard batches had been rejected without a compensating number of high ones being rejected.

With regard to the first reason, Mr Giles agreed that the Author might be right, supposing that the number of times the cement over-ran was neither too many nor too few, for in the first case the errors in batching cement reverted to being truly random and in the second case they would have had negligible effect.

Turning to the rejection of sub-standard batches, Mr Giles supposed that even if a sample should show a batch as being sub-standard, and even if the batch was rejected, the sample result should still be recorded to show the true range of concrete strengths actually produced at the batching plant. That conformed with the theory that there would be other bad concrete in batches which had not had the misfortune to be sampled.

In the correspondence on Paper No. 5879 [4] Mr Giles had given what he considered to be an explanation of asymmetry in histograms resulting from concrete sampling. He considered that on the job they represented the difficulty with cement excesses had not been met.

The Author, in reply, observed that both during the progress of the works described, and when subsequently writing the Paper, he had been very conscious that works cube test results were almost useless unless and until they were correlated reliably with inspection and control observations during production. Mr Sivewright had referred to the investigations as an academic exercise, and had gone on to ask what happened when results were below the limits fixed. In fact, the purpose of the analyses in the first instance had been the entirely practical one of finding out why variation was high and whether it could be reduced.

By identifying the factors causing variation, and assisting estimates of their relative effects, the methods used had indicated where and how improved control and inspection could be applied, and as a result the required standards had been achieved more economically than would have been possible otherwise. Works cube tests should be regarded as the ultimate and drastic deterrent to any carelessness in production: once the results achieved under an observed range of typical conditions had been established in terms of cube strength during preliminary tests or the initial stages of the job, the engineer's task was to prevent the conditions deteriorating. In the Paper, prominence had been given to cube results because they were the best available measure of the results achieved, and because they might be useful in future work; but the success or failure of quality control lay in the establishment and enforcement of rational standards of regulation at the batchers and mixers.

In spite of the best efforts in control and inspection, cube results might infringe the required standards in two ways. Either a whole series of results might show a gradual reduction in average strength or a gradual increase in variation, so that too high a percentage began to fall outside the "warning limit," or within a series showing normal average and scatter an individual result might suddenly be obtained below the "acceptance limit." In the first case, a series of intensive tests to establish and correct the downward trend would be called for; if such tests indicated that control could not be brought back to the original level, the only course would be to raise the average strength by enriching the mix. Such intensive testing had been called for at Woodhead after the first series of results, with the results described, but subsequently no significant trend had been observed. In the second case, that of isolated low values, sufficient samples would have to be cut from the section concerned to establish both the average and the range of strengths in the work, and a careful appraisal

[4] N. M. Plum, "Quality Control of Concrete—Its Rational Basis and Economic Aspects." Proc. Instn Civ. Engrs, Part I, vol. 2, p. 311 (May 1953). *Correspondence* by E. R. Giles: *Ibid.*, vol. 3, p. 100 (Jan. 1954).

would be made from those results of the actual factor of safety of the structure.

At Woodhead there had been one case of significantly low strength; in this instance the net factor of safety was estimated to be adequate and the cutting out of the lining arch was not ordered.

It was true that the use of 28-day strength as the final criterion resulted in a time-lag in the assessment of samples, but with a reasonably close standard of control, and continuous cement and aggregate testing, any downward trend would not be so rapid as to do much harm before it was shown up. As for the discovery of an isolated bad region, it made little difference in the particular case of a tunnel lining whether the result was known hours or weeks after placing: once the concrete was in it would not be practicable to cut it out for a considerable time, even if its fate were known immediately. On other types of work a quick answer might be of great importance, and non-destructive tests in situ might find an application.

On the subject of the cost of tests, it was not possible to give any helpful figures, but it was worth recording that the study described in the Paper had been carried out on site without any increase in the resources originally intended to cover concrete testing on conventional lines. Testing costs had been, and should always be, an extremely small fraction of total expenditure, but there was an important balance between the cost of batching and placing plant and the economies in cement which could be effected by their improvement.

It was gratifying that Mr Sivewright had accepted and underlined what might be regarded as the most important conclusion of the Paper, that was, that full-scale field trials should be made at the beginning of every job, in order to co-ordinate the whole task of planning and producing a material to meet the requirements of the designer with the actual plant and materials available. Not everyone accepted that idea, as might be seen from the correspondence on Dr Murdock's Paper,[5] but it was beginning to be adopted.

Perhaps the point about excess cement had not been made clear, for Mr Giles appeared to have misunderstood it. The mechanics of the batcher operation produced virtually no batches with less cement than intended, a majority with the correct amount, and some with various percentages of excess. That alone would result in a completely skew strength-histogram with no results less than the mode. Cube results had been recorded only from concrete actually put into the work, so that the rejection of "wet" batches had further biased the histogram. The interesting displacement to which Mr Giles had called attention might well have operated, but was small compared with the effects mentioned in the Paper.

[5] L. J. Murdock, "The Control of Concrete Quality." Proc. Instn Civ. Engrs, Part I, vol. 2, p. 426 (July 1953); *Correspondence, Ibid.*, vol. 3, p. 233 (Mar. 1954).

OBITUARY

HUGH RICHIE BARR, who died at Aberdeen on the 30th October, 1954, was born on the 18th September, 1876.

He was educated at Perth Academy and at the Technical College, Dundee.

He served his apprenticeship with the Caledonian Railway Co., subsequently becoming Assistant Engineer to the Company.

In 1903 he joined the staff of the Aberdeen Harbour Commissioners, later becoming Harbour Engineer, Aberdeen.

For a number of years he acted as Consulting Engineer to Fraserburgh Harbour Commissioners.

Mr Barr was elected an Associate Member in 1905.

He is survived by his wife.

FREDERICK FRANCIS PERCIVAL BISACRE, O.B.E., M.A., B.Sc., F.P.S., who died at Helensburgh, Scotland, on the 9th November, 1954, was born on the 20th June, 1885.

He was educated privately and at Trinity College, Cambridge.

From 1910 to 1919 he was with Merz and McLellan, Consulting Engineers, first as Assistant, and later as Personal Assistant to Mr Charles Merz.

In 1920 he joined Blackie & Son Ltd, and became a Director, and subsequently Chairman of the Company.

Mr Bisacre was elected an Associate Member in 1914.

For his Paper on "Overhead Track Construction for Direct-Current Electric Railways," [1] he was awarded a Crampton Prize.

He is survived by his wife, two sons, and one daughter.

SOMERS HOWE ELLIS, F.R.G.S., who died on the 8th October, 1954, was born on the 25th February, 1871.

He was educated at King's College London.

He began his professional career as Assistant Engineer in the Construction Department of the former Midland Railway Company.

In 1897 he was appointed Assistant Engineer on the East Indian Railway. In 1903 he became Resident Engineer on the construction of the Tranmere Bay Development Works for Cammell, Laird & Co., at Birkenhead.

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[1] Min. Proc. Instn Civ. Engrs, vol. 208 (1918–19, Pt II), p. 418.

Between 1906 and 1919 he was in private practice, but most of his work was commissioned by Alfred Holt & Co., who were at that time developing their own wharf properties in Hong Kong and Shanghai. The port facilities available were inadequate to the increasing volume of cargo and size of ships employed in the trade. The wharf at Shanghai—nearly half a mile long—presented particular problems, for the Yangste Delta consists of several hundred feet of soft silt and all the structures had to be supported on skin-friction piles or on rafts.

During World War I he developed similar properties for the Company in the Netherlands East Indies with equal success and in July 1919 was appointed Chief Civil Engineer to the Company in Liverpool, a post which he held with distinction till he retired in May 1939. Apart from the design and equipment of port facilities to meet the varied requirements of ships and cargoes, he also had much to do with the large office buildings erected by the Company in Liverpool and Singapore in the 1920's.

Mr Ellis was elected an Associate Member in 1896, and was transferred to the class of Members in 1907.

He was awarded a Telford Premium in 1908 for his Paper on "The Tranmere Bay Development Works,"[1] and a Manby Premium in 1912 for his Paper on "Reinforced Concrete Wharves at Lower Pootung, Shanghai."[2] He also contributed a Paper on "Corrosion of Steel Wharves at Kowloon."[3]

He is survived by his wife, a son, and two daughters.

FRANK HARDING JONES, who died in London on the 9th July, 1954, was born on the 3rd September, 1873.

He was educated at Rugby and received his early training during the construction of Tower Bridge.

After working in his father's consulting engineer's practice, he was subsequently appointed Director of the South Metropolitan Gas Company, and in 1937 became Chairman.

He was also concerned with water and hydraulic supply companies.

Mr Jones was elected a Member in 1909.

He is survived by his wife, a son, and three daughters.

HENRY NIMMO, C.B.E., who died on the 25th October, 1954, was born on the 28th July, 1885. He was educated at Airdrie Academy and Coatbridge Technical College.

From 1900 to 1906 he obtained practical training in mechanical and heating engineering. From 1907 to 1912 he was Chief Electrical Engineer, first with the Oakbank Oil Company and subsequently with the Irrawaddy Flotilla Company in Rangoon.

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[1] Min. Proc. Instn Civ. Engrs, vol. 171 (1907–08, Pt I), p. 127.
[2] Min. Proc. Instn Civ. Engrs, vol. 188 (1911–12, Pt III), p. 80.
[3] Min. Proc. Instn Civ. Engrs, vol. 199 (1914–15, Pt I), p. 133.

Mr Nimmo was Assistant Engineer to the Rangoon Electric Tramway and Supply Company from 1912 until 1914. From 1914 to 1915 he was Electrical Inspector and Adviser to the Government of Burma.

He served in the Royal Engineers from 1916 until 1920 and rose from sapper to the rank of Captain in the Electrical Branch.

During the years 1920 to 1929 he served with the Government of Burma, ultimately becoming Chief Electrical Inspector and Adviser.

From 1929 to 1945 Mr Nimmo was Chief Engineering Inspector to the Electricity Commission and in 1945 was appointed Electricity Commissioner. He had been Chairman of the Southern Electricity Board since 1948.

Between 1951 to 1954 he was Vice-President and Member of Council of Conférence Internationale des Grands Reseaux Electriques à Haute Tension and from 1949 to 1954 was a Member of Council and Chairman of Union Internationale des Producteurs et Distributeurs d'Energie Electrique.

In connexion with the 1941 Ingenuity Competition, he presented to the Institution a short Paper,[1] which was later published in the Institution Journal.

Mr Nimmo was elected a Member of the Institution in 1936. He was also a Member of the Institution of Mechanical Engineers and the Institution of Electrical Engineers.

He was made a C.B.E. in the Coronation Honours of 1953.

He is survived by his wife and 2 daughters.

LT-COL. KENNETH MAURICE STEVEN, B.Sc., who died on 18th October, 1954, was born on the 2nd May, 1905.

During the 1939–45 war he served as a volunteer from Argentina with the Royal Indian Engineers, first in Ceylon and later as Assistant Director of Transportation (Railways) with the South-East Asia Command. Upon leaving the Indian Army he was granted the honorary rank of Lieutenant-Colonel.

In 1946 Colonel Steven joined the Nigerian Railways, as district Engineer and was transferred in 1950 to Gold Coast Railways as Chief Engineer. He was appointed Deputy General Manager, Gold Coast Railways, in 1952 and held this position at the time of his death.

Colonel Steven was elected an Associate Member of the Institution in 1931, and was transferred to the class of Members in 1948.

He is survived by his wife and two children, a son and a daughter.

THEODORE STEVENS, E.M., B.Met., who died in hospital at Herne Bay on the 18th October, 1954, was born on the 2nd April, 1865. He graduated from Lehigh in 1886.

[1] "The Rewinding of a Stator at Rangoon, Burma." J. Instn Civ. Engrs, vol. 17, p. 386 (Feb. 1942).

From 1887 to 1892 he was engaged in development work on the electrolysis of aluminium fluoride for the Aluminium Company of America. Then from 1893 to 1898, as Works Manager of the Thames Iron Works & Shipbuilding Company, he was in charge of the manufacture of electrical machinery, including locomotives.

In 1898 Mr Stevens joined the British Thomson Houston Company and was in charge, as Chief Engineer, of the electrification of the Central London railway.

From 1912 until his retirement he practised as a consulting civil, electrical, and mining engineer. Schemes with which he was concerned included hydro-electric developments in Argentina, electrification of the São Paulo railway (Brazil), the Aswan Dam, investigations into the water-power resources of Palestine and Ireland, the Channel Tunnel, and the London-Paris Railway. Later he sat on the Board of the Margate and East Kent Electricity Company.

He was joint author of a book on "Steam Turbine Engineering" which was published in 1906.

Mr Stevens was elected an Associate Member of the Institution in 1904, and was transferred to the class of Members in 1910. He was also a Member of the Institution of Electrical Engineers.

He is survived by his wife, one daughter, three grand-daughters and one great-grand-daughter.

GEORGE BRANSBY-WILLIAMS, who died at his home in Cooden, near Bexhill, on the 17th November, 1954, was born at Swansea on the 7th April, 1872.

He was educated at Clifton College, and in 1891 was articled to James Mansergh (then Vice-President I.C.E.). In 1895 he was appointed Resident Engineer on the Birmingham water-supply aqueduct and served in various capacities on that project until 1900. Then followed two years in South Africa serving in the 3rd Glamorgan Battalion, Welch Regiment, in charge of the repair, maintenance, and reconstruction of railways, bridges, and stations.

From 1902 to 1906 he was assistant to Mr G. R. Strachan, M.I.C.E., and was engaged on numerous sewerage and water-supply schemes in East Africa. Then followed three years as Chief Engineer to the Public Health Department, Bengal, and from 1909 to 1927 he was head of the Department.

Mr Bransby-Williams returned to England in 1927 and became consulting engineer in the firm of Williams, Temple and Bartholomew, but his activities were still concerned with major public health schemes in India, and later Burma. He retired in 1945.

He was author of several technical books ; Elementary Sanitary Engineer ; Sewage Disposal in India and the East ; The Flow of Water ; Storm

servoirs; and numerous technical Papers, three of which were published the Institution.[1, 2, 3]

Elected an Associate Member in December 1897, Mr Bransby-Williams s transferred to the class of Members in February 1909.

He is survived by his wife and two sons.

[1] "Lining a Reservoir near Whitby." Min. Proc. Instn Civ. Engrs, vol. 137 98–99, Part III), p. 357.

[2] "Repairs to and Maintenance of the Pretoria Eastern Railway." *Ibid.*, vol. 148 1906–7, Pt II), p. 276.

[3] "Rainfall, Off-Flow, and Storage in the Central Provinces, India." Selected gineering Paper, No. 106. Instn Civ. Engrs, 1931.

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